1 General

1.1 Contents

This memorandum deals with the geometric design of rural roads and junctions and supersedes Memorandum 575 (Layout and Construction of Roads)[1] and Memorandum 780 (Design of Roads in Rural Areas).[2] It does not cover aspects such as traffic growth and road construction; the former is now dealt with in a companion memorandum:* little reference is given herein to traffic signs, and reference should be made to the Traffic Signs Manual. (H.M.S.O.)[4]

The standards are those which should be applied to new construction or, as far as practicable, in the improvement of existing roads. They do not of themselves provide justification of improvement.

The standards given herein relate to roads in rural areas. For intermediate conditions, and for Urban roads, reference should be made to the Manual on Roads in Urban Areas[5] and to the Memorandum on Urban Traffic Engineering Techniques. (H.M.S.O.)[6]

1.2 General Considerations

Roads and junctions must be designed to provide adequate traffic capacity for the future and safety for both existing and future conditions. From the safety aspect, as about 42% of serious accidents occur on straight sections of road, 32% at bends, and 26% at junctions, balanced attention must be given to all parts of a road design. With regard to capacity, whilst this is generally limited by road junctions, the effect of inadequate roads is to delay traffic, and this consequently results in high economic losses. It has been estimated that the loss from road accidents alone was about £200 million in 1963, and losses due to congestion were even higher.

1.3 Traffic Information

The satisfactory design of both roads and junctions can only be made when adequate traffic information is available. The information obtained must be sufficient to enable a forecast of traffic flows to be made, and is based on traffic censuses or Origin and Destination Surveys. The estimated traffic volumes must be scaled up in accordance with current forecasts of traffic growth for the period for which the scheme is designed: details are given in the Manual on Traffic Prediction for Rural Roads.[3] Whilst the general aim should be towards providing the capacity required in the long term, it will often be more economic to plan for a shorter period, but any doubts on the wisdom of a particular choice should be cleared by making an economic assessment. For the directional distribution of traffic at junctions, unless known factors will change the traffic pattern, it will generally be necessary to assume a similar distribution in future years to that derived either from current traffic counts or Origin and Destination surveys.

*Manual on Traffic Prediction for Rural Roads[3]

1.4 Design Criteria

1.40. Roads between junctions.

The basis for rural road designs is the average daily flow (7 day average 6 a.m.–10 p.m.) taken in August,* with an addition for future traffic as outlined in Section 1.3. If the traffic census has been taken at some time in the year other than August, the August flow in p.c.u's can, in the absence of local data, be estimated by means of the multiplying factor given in the third column of Table 1–40. As a general rule, traffic counts taken during the months of November to March should not be used to determine the August level. The traffic flow will need to be converted into passenger car units—see Section 1.42. See Section 3.20 for road capacity.

Table 1–40 Seasonal Variation in Traffic Flow (p.c.u's)

Month	Percentage Average Month	Multiplier to give August flow
January	71†	—
February	77†	—
March	88†	—
April	100	1.26
May	108	1.17
June	116	1.09
July	123	1.03
August	126	1.00
September	115	1.10
October	101	1.25
November	90†	—
December	84†	—

†Very variable.

*The August flows should not be taken during the Bank Holiday.

1.41. Road junctions. Junction design should be adequate to accommodate the heaviest traffic flow conditions normally occurring. The traffic flow is usually derived from a directional census taken in August, with an addition for future traffic as outlined in Section 1.3. Adjustment for censuses taken at other suitable times of the year can be made as given in Section 1.40: on holiday routes the censuses should be taken at times when maximum flows are expected.

The traffic flows will need to be converted into the appropriate passenger car units for the particular type of junction design. (See Table 1–42.)

The heaviest traffic flow conditions usually occur at peak times but the design needs to be tested for combination of flows at other than peak times. It is also necessary to check that the design of a junction will cater for the various conditions occurring throughout the week. However junctions should not be expected to cater for exceptionally heavy flows which infrequently occur, and censuses should not normally be taken when these flows can be expected. The census desirable to achieve the above objectives is a four day hourly directional count taken in August (or other busiest month), on Friday, Saturday, Sunday and Monday. Further censuses are often advantageous, but may be limited to the day and hours which have been found by the first census to be critical.

1.42. Passenger Car Units. In order to allow for variations in the effect of different types of vehicles upon the capacity of roads and junctions, traffic volumes are expressed in passenger car units (p.c.u's). The basic unit is a car—other light three and four wheeled vehicles are also allotted one unit. As different types of vehicles affect the capacity of rural roads, urban roads, roundabouts, and traffic signal controlled junctions in varying degrees the weighting of each class of vehicle differs according to the purpose for which they are used. Appropriate values are given in Table 1–42.

Table 1–42 Vehicular Ratings in Passenger Car Units

Class of Vehicle	Equivalent value in passenger car units (p.c.u's)			
	Rural Standards	Urban Standards	Round-about Designs	Traffic Signal Designs
Private cars, Motor Cycle Combinations, taxis and light goods up to 30 cwt. unladen	1.00	1.00	1.00	1.00
Motor Cycles, (solo) Scooters and Mopeds	1.00	0.75	0.75	0.33
Goods Vehicles over 30 cwt. unladen, and horsedrawn vehicles	3.00	2.00	2.80	1.75
Buses, Coaches, trolley vehicles and trams	3.00	3.00	2.80	2.25
Pedal Cycles	0.50	0.33	0.50	0.20

2 The Road System

2.1 Communication Systems

The existing pattern of rural roads has grown to meet the local needs of towns, villages and the rural area, and is seldom satisfactory for the changed nature of present day traffic and the increased number of long-distance journeys. In consequence the road pattern is seldom efficient in linking the principal areas of traffic generation as well as providing for local traffic needs.

At the higher levels of demand the motorway system meets the situation, but at lower levels simpler solutions have to be found, and it is not generally practicable to provide a system of primary routes divorced from the existing road layout. It is advantageous, however, to consider how some adaptations can be made towards a similar result. In such a system spur roads would be used to link urban areas to the new routes; one spur only would be needed for a village, and this system would take the place of the normal one of by-passing urban areas.

2.2 Road Siting and Amenities

In choosing the alignment of a new road, or the improvement of an existing one, regard should be given to choice of gradients and curvature to provide smooth flowing and economic transport. It will generally be possible to satisfy both these requirements, e.g. a route of longer distance may be preferable to a shorter one involving sharp gradients or high altitudes where fog and snow may cause difficulties. In general, long curves are preferable to straights, and the lengths of curves on a given road should be of similar order. For aesthetic reasons changes in vertical and horizontal alignment should generally be phased to coincide where possible; it is, however, necessary for safety reasons to avoid short vertical curves. Short straights between pairs of horizontal or vertical curves which are visible one from the other are also aesthetically undesirable: it is better to introduce a flat curve between curves of the same sense or to extend curves of the opposite sense to a common point. There is no easy way of ensuring a satisfactory aesthetic design and the use of models and graphical methods should be considered for important projects. It is important where practicable to preserve existing amenities and if possible to make use of them to enhance the road layout, e.g. preservation of groups of trees, attractive buildings.

The designs and siting of bridges should fit into the general road alignment. As, however, it will often be necessary, on account of local topography or nature of foundations, for the siting of a bridge to be fixed, the road will need to be located so that a natural flowing alignment is obtained.

Long curved bridges are usually costly, and bridges with large skews are particularly so and should generally be avoided: small bridges, however, should be unobtrusive and fit in with the general road alignment, curved as necessary.

On curves of small radius minimum sight line requirements in Design Table II may necessitate the setting-back of side piers (or abutments) of overbridges and either off-setting the central pier, with consequential widening of the median, or the omission of the central pier. In extreme cases these measures can increase the cost of bridgeworks by as much as 40%, but they are also of value in reducing collisions with the bridge structure. Additional bridgework costs, especially in the case of skew bridges, are also occasioned when overbridges on curves are sited less than the minimum sight-line distance apart.

In cases where compliance with sight line requirements results in increased bridgework costs alternative roadwork layouts should be considered so that a solution involving the minimum combined cost of bridgeworks and associated roadworks is adopted. While there should be no general relaxation in sight-line requirements a reduction over a limited length may exceptionally be permitted when it can be shown that this will produce a worth while saving in capital cost.

Where it is practicable to re-align approach roads so as to avoid skew crossings an assessment should be made of the additional roadworks and land costs and the choice between a straight or skew crossing made on this evidence. In those cases where a number of skew crossings on a contract length of road are unavoidable the maximum practicable standardisation of skew should be adopted throughout the contract. In some cases it may prove economical to provide slightly increased spans so as to achieve uniformity with those at sites at which a greater degree of skew has to be provided.

Where roads are in cut, the appearance of the slopes requires special attention and it is desirable, if possible, that natural conditions be simulated. This can sometimes be achieved by extra excavation at the beginning and end of the cut, but the extent to which this should be done must depend on economic considerations. Appearance is also enhanced by rounding off slopes: to do this effectively usually involves considerably more land and can only be done in cases where land is cheap or arrangements can be made so that little extra land needs to be taken inside the highway limits. It is desirable that as much land as possible should be returned to agriculture, but in most cases flatter slopes will be necessary if this is to be done.

The siting and choice of suitable trees is important and they should never interfere with visibility or be of types likely to outgrow their position. Any planting should not be used as applied ornament but should have a definite purpose such as the restoration of landscape following road construction, or the screening of unsightly features. Normally only indigenous species which occur in the area should be planted and any planting must be informal and related to the existing landscape pattern. Where shrubs are planted in the central reserve they should not encroach within 4 ft. of the carriageway (see also Section 3.15); large trees should not be planted on heavy clays within about 40 ft., or on other soils within 12 ft. of the carriageways, lay-bys or hard strips.

2.3 Traffic Management

The rapid growth of traffic makes it essential to obtain the best use of existing roads and valuable improvements in safety and traffic flow can be gained by the application of some traffic management techniques.

The scope for certain of these techniques such as one-way working and banning of right turns is limited by the larger scale network of rural roads which would make the detours resulting from the restrictions excessive. In considering such schemes the gains to be obtained must be weighed against the disadvantages.

A general aim should be towards eliminating standing vehicles which are a cause of accidents as well as an impediment to traffic. Clearway standards are desirable for all important roads and to apply these standards it is necessary to provide standing areas off the carriageway. Even where clearway standards are not applied an adequate provision of parking areas, lay-bys and bus lay-bys is desirable (see Section 3.16).

Right turns and U turns are also a danger and reduce traffic capacity and reference is given in Section 3.44 to methods which may be employed to meet this problem. Substantial reductions in accidents are obtained by minimising the number of road junctions on a road and an example is given in Section 4.21.

Driver comprehension and usage is important and the provision of traffic signs and carriageway markings improves the orderly and safe flow of traffic; reference should be made to the Traffic Signs Manual[4] for the signs and road markings to be employed.

Where roads pass through development, the methods of obtaining improvements in traffic flow and safety given in Urban Traffic Engineering Techniques[5] generally apply and there will often be scope for restriction on waiting and on the loading and unloading of vehicles.

3 The Road between Junctions

Cross Sectional Elements

3.10 The Carriageway

The standard lane width for dual carriageways and two lane single carriageways is 12 ft.; this width is exclusive of that required for traffic islands, hardened strip or central reservations. Where exceptionally, a three lane road is adequate for future traffic, the standard lane width should be 11 ft.

It should be noted, however, from Table 3–20 that light volumes of traffic can be accommodated on single carriageway two-lane roads of lesser overall width than 24 ft. There is some evidence, however, that accidents increase on narrow two lane roads particularly if less than 18 ft. wide, which should be regarded as the minimum.

Single lane roads and passing bays are sometimes appropriate where traffic volumes are very light: a width of 11 ft. is desirable and passing bays should be provided at frequent intervals of about 8–10 per mile. The siting should be arranged so that from any point on the road at least one bay is visible and from any bay the bays on each side can be seen. On sharp bends a two lane carriageway is desirable.

3.11 Verges, hard shoulders and footways

Design Table I sets out recommended treatment for roads of differing importance. For slip roads, single-track roads, etc., a minimum verge width of 6 ft. is desirable. In addition added width will be needed for cycle tracks and footways where required, and sometimes for the accommodation of traffic signs and the visibility splays thereto.

Verges should be sufficiently even to permit their occasional use by pedestrians or for the storage of snow displaced from the carriageway.

The widths given may be reduced as indicated under bridges or where localised restrictions prevent the full width being obtained, see Design Table I, fourth column.

Footways will not usually be needed except in the vicinity of villages and they should be set back as far as practicable from the carriageway edge, and if possible not less than 4 ft. The minimum width of footway for regular use should be 5 ft.; for infrequent use a narrower footway may suffice. Where hardened strips are provided, the footway should always be sited as close to the highway boundary as possible.

3.12 Kerbs and Edgings

The edging to carriageways should be marked either by edge lines or by splay, half batter, flush, lip or vertical kerbs. Continuity and uniformity should be the general aim but the choice will depend upon particular circumstances. Table 3–12 indicates recommended standards.

Design Table I

Reference No. (1)	Type of Road (2)	Recommended Kerbside Treatment* (3)	Minimum Kerbside Treatment*† (4)
1	Dual carriageway and Three-lane single carriageway roads.	12 ft. verge comprising edge lining on hard strip 3–4 ft. wide and remaining width grassed.	6 ft. verge comprising edge lining on hard strip 3–4 ft. wide and remaining width grassed.
2	Two-lane single carriageway roads with design year flows exceeding 6000 p.c.u's/day.	12 ft. verge comprising edge lining on hard strip 3–4 ft. wide or kerbs and remaining width grassed.	6 ft. verge comprising edge lining on hard strip 3–4 ft. wide or kerbs and remaining width grassed.
3	Two-lane single carriageway roads with design year flows between 3000 and 6000 p.c.u's/day.	12 ft. grass verge: edge lining or kerbing not usually required.	6 ft. grass verge: edge lining or kerbing not usually required.
4	Two-lane single carriageway roads with design year flows under 3000 p.c.u's/day.	6 ft. grass verge: edge lining or kerbing not usually required.	4 ft. grass verge: edge lining or kerbing not usually required.

†On bridges, surfacing will replace grass. (See also Section 3.40.)

*See also Table 3–12.

Table 3–12 Kerbs and edge lines

Type of Road	Kerbing or Edge Lining
Dual carriageway roads and three-lane roads.	6 in. wide reflectorised line* on hard strip 3–4 ft wide on both nearside and offside of carriageways. Only where needed for constructional reasons should flush kerbs be used additionally. Reference should be made to the Traffic Signs Manual[4] on the uses of reflecting studs.
Two-lane roads with grass verges, of principal or trunk road status, or roads with heavy, night traffic, usually with existing flow exceeding 3000 p.c.u's/day.	(a) 6 in. wide reflectorised line* on hard strip 3–4 ft. wide. Only where needed for constructional reasons should flush kerbs be used additionally. (b) 45° splay kerbs with reflectorised line.
Two-lane roads with grass verges, of less importance, usually with existing flow below 3000 p.c.u's/day.	Kerbing or edge lining not normally required, but reflectorised lines may be needed, particularly on bends or where accidents occur.
Two-lane roads with footways alongside.	4 in. high half-batter kerbs with reflectorised lines.
	*8 in. wide marginal strip of calcined flint surfacing may be used as an alternative.

(a) *Edge Lines.* These lines, where used in substitution for kerbs should consist of either a continuous 6 in. wide line of reflectorised paint or thermo plastic, or an 8 in. wide inset of calcined flint. It is necessary to provide a hardened strip with the edge line 3 to 4 ft. in width and also to ensure that drainage is effective, see Section 3.13. Edge lines have the merit that resurfacing of a road can be carried out more economically than where kerbing is used.

(b) *Splay Kerbs.* 45° splay kerbs laid with 4 in. upstand are suitable where it is impracticable to provide the hardened strip required for edge lines or flush kerbs. They are appropriate also for the demarcation of traffic islands, roundabouts, and along roads where a verge separates the carriageway from a footway or cycle track.

(c) *Half batter kerbs* are normally required only where footways are less than 4 ft. from the carriageway.

(d) *Lip kerbs.* These kerbs have an upstand of about 2 ins. and may be formed by the use of half batter kerbs laid flat.

(e) *Vertical kerbs* are not generally recommended but may be used to reduce the risk of vehicles leaving the carriageway in circumstances where a safety fence is not justified (see Section 3.17).

(f) *Flush kerbs.* Where flush kerbs are used they should always be used in conjunction with a reflectorised line unless a calcined flint surface has been incorporated. A hardened strip and the provision of effective drainage are needed as for (a) above. Rippled or corrugated flush kerbs are more conspicuous at night than plain flush kerbs and the rippling serves to warn drivers when they are tracking the kerb. Flush kerbs should normally only be used instead of edge lines where required for constructional reasons.

(g) *Special kerbs,* e.g. incorporating a drainage channel may be used with advantage in some circumstances.

(h) *Edgings.* At the back of hard strips 3 in. upstand splayed edgings, either of precast concrete or of extruded construction are recommended, where positive drainage is required on embankments.

3.13 Surface Water Drainage

Road drainage must be effective not only to clear the road of surface water and prevent it from seeping into the road foundation but also to prevent mud from the verge, or grit from the road, obscuring the edge line or kerb to the carriageway, and also to prevent any adverse effect from existing land drainage. Water and drainage authorities will need to be consulted to ensure their interests are safeguarded. Advice on rainfall intensities may be obtained from the Meteorological Office.

Surface water drainage systems should normally be designed to carry a 1-year storm and reference may be made to Road Note No. 35, a guide for engineers to the design of storm sewer systems, London 1963 (H.M.S.O.).[7] This note gives recommended procedures for methods of design. Where the consequences of flooding are likely to be very serious, or for watercourses, the design may be based on heavier storms.

The type of drainage used will depend on the type of carriageway edging, the subsoil and whether the road is in cut or on embankment. A minimum longitudinal fall of 1 in 250 is recommended, and special attention is required at changes from superelevation to normal camber to ensure longitudinal fall.

Where positive drainage is required on roads where edge lines are used instead of kerbs, or along lengths of adjacent lay-bys or parking areas, french drains may be used or alternatively upstanding kerbs or channels with connections to side drains or ditches. The use of french drains is more suitable in cuttings, where they should be placed behind the hard strip see Fig. 3–10. On embankments, an upstanding edging, or a channel may be used at the rear of the hard strip or apron to the lay-by or parking area, with connections to the ditch or side drains by means of untrapped gullies and pipes or by channels. Where french drains are used for the collection of surface water, catchpits with open gratings should be provided not more than 100 yards apart.

Positive drainage will normally only be needed on the more important roads and/or where the subsoil is such that simple grips to ditches may lead to weakening of road foundations.

On less important kerbed roads the drainage may be effected by breaks in the kerb line and drain connections to ditches or 'grips'.

Central reserves should be effectively drained by the use of french drains and connections at intervals across the carriageway to surface water drains or watercourses.

3.14 Cycle Tracks

As the volume of cycle traffic in rural areas has fallen by over 10% a year in recent years, cycle tracks are rarely justified. Cycle tracks may, however, be considered on safety grounds where the volume of cycle traffic exceeds 1,000/day, particularly where there is a local demand between residential and factory areas, but regard should be given to the economic benefits that would be obtained, particularly where the cost of providing cycle tracks is high. There may be circumstances where a cycle way should be considered to serve a local need.

Cycle tracks should normally be designed for one-way traffic though two-way operation may sometimes be advantageous. For one-way the minimum width should be 6 ft. For peak flows above 500 cycles per hour the width should be increased by 3 ft. for each additional 500 cycles per hour. For two-way operation the normal width should be 12 ft.

Cycle tracks should have a well drained good riding surface, gently ramped to join the carriageways and passing without interruption across vehicular entrances. It is desirable that a cycle track should be separated from the carriageway by a 6 ft. verge, and if a footway is provided, it should be behind the cycle track and separated from it by a verge 3 ft. wide. Where a reduction in

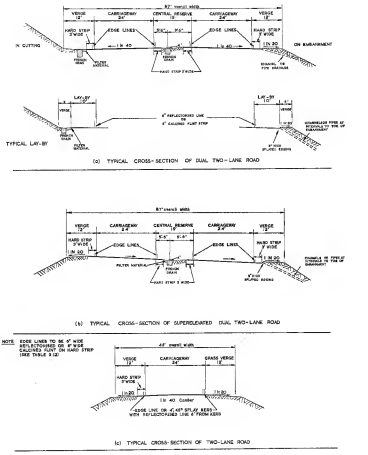


Fig. 3.10 Cross-Sectional Layout of Roads

width is necessary, the width between the cycle track and the carriageway should be preserved as far as possible and should not be less than 4 ft.

At busy single-level junctions with a high volume of cycle traffic, the construction of cycle subways may be warranted: these may sometimes be combined with pedestrian subways.*

3.15 Central Reserve

There are advantages to be gained from wide central reserves but for economic reasons the width must be restricted.

The basic width of a central reserve between road junctions is 15 ft. (inclusive of marginal strips or kerbs) but may be reduced to 10 ft. where conditions (geographical, agricultural, etc.) prevent the desired width being obtained. The width of a central reserve may be reduced below the 10 ft. width given above for special conditions. A central reserve width of 6 ft. is satisfactory on via-ducts or long bridges, but for any circumstances the width should not be less than 4 ft., and this absolute minimum should only be adopted over short lengths, e.g. where valuable or historic buildings are close to a road. Widths of more than 15 ft. may be required at junctions (see Section 4), accesses, for landscaping, on side long ground (where carriageways may be on different levels), for duplicating existing road, or for extra safety where land is cheap, etc. Central reserves should be slightly dished so that water, melting snow, etc., will not encroach onto the carriageways. Shrubs, either grouped, or continuous, are useful for screening headlight glare as well as for amenity. In planting shrubs regard must be given to providing adequate clearance between them and the carriageway (about 4 ft.), and to ensure that visibility standards are maintained along the road. Owing to the use of salt for de-icing, shrubs may be difficult to establish unless the width is adequate or the reserve does not take water from the road surface. Where an additional carriageway is constructed alongside an existing road, boundary hedges and trees should be preserved, but the normal visibility standards must be provided for both carriageways.

On long sections of road with a continuous central reserve, reserve crossings should be provided for the purpose of diverting traffic from one carriageway to the other at times of emergency or when major repairs are taking place. Such crossings should be 55 ft. long with radii to the central reserve of 200 ft.; rounded off to bullet ends of about 1 ft. radius. They should be provided near major junctions and at approximately two mile intervals: they should normally be closed by barriers which can be easily removed when necessary.

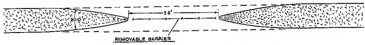


Fig. 3–15 Central reserve crossing

3.16 Lay-bys and Parking Areas

The aim should be to provide clearway standards by the elimination of any standing vehicles on carriageways. Whether or not the road is designated as a clearway, an adequate number of lay-bys and/or parking places is an essential feature of the design; they serve as halting places on long journeys, for broken down vehicles, for the enjoyment of places of scenic interest, or for parking whilst making telephone calls.

Continuous paved hard shoulders will seldom be justified, but it is recommended that two lengths of lay-by per mile, each about 100 yards long, should be provided on each side of three-lane and dual carriageway roads (see Fig. 3–16B). Between the lay-bys there should be a hard strip 3–4 ft. wide behind the edge of the carriage-way, with 1 in 20 tapers between the 10 ft. wide lay-bys and the hard strips.

The half mile spacing of lay-bys may be varied to some extent for topographical reasons and to suit horizontal and vertical alignment, e.g. they should preferably not be sited on curves or crests, but may advantageously be located at or near the bottom of gradients. They should always be sited away from bridges or other structures, and it is advantageous for their surfacing to be distinctive from that of the carriageway.

The siting of parking areas will depend very greatly on local conditions and the availability of land: sometimes areas of disused carriageway can be used as a parking place. On single carriageways, they are best sited at approximately equal intervals alternately on each side of the road. To discourage drivers from turning right into an offside parking area, advance signs for the next nearside parking area (and toilet) should always be provided. Lay-bys and parking areas should not be sited near junctions or where visibility is restricted. Typical layouts are shown in Fig. 3–16 (A–D).

For dual carriageway and three-lane roads parking places separated from the carriageway as in A may also be provided, spaced at about 6 mile intervals on each side of the road; closer spacing may be needed on roads attracting holiday traffic. For two-lane roads carrying more than 6000 p.c.u's/day, types C or D at approximately 1 mile intervals on each side of the road will be appropriate. For flows between 3000 and 6000 p.c.u's/day, Type D at 3–5 mile intervals should be provided, and for flows under 3000 p.c.u's/day Type D, occasionally as needed.

Parking areas should be hardened and adequately surfaced preferably distinctively from that of the main carriageway. Where space permits or where heavily used, for picnicking, etc., on holiday routes, it is desirable for the parking area to be away from the main carriageway as in A or divided from the main carriageway by a verge as in layout C. Conveniences should be sited at suitable

*Roads in Urban Areas (H.M.S.O.)[5]

places, not close to houses, preferably at adjacent lay-bys on each side of the road.

Standard lay-bys or parking areas are not intended for bus stopping places which should usually be in separate lay-bys so as to be as convenient for passengers as possible, consistent with safety for the bus and other road users. There is no objection to suitable lay-bys or parking areas serving a dual purpose provided the bus stopping area is appropriately delineated to prevent other vehicles stopping on it. Typical recommended layouts are given in Figs. 3–16E and 3–16F.

Restrictions on parking may need to be considered to facilitate movements to and from the parking area.

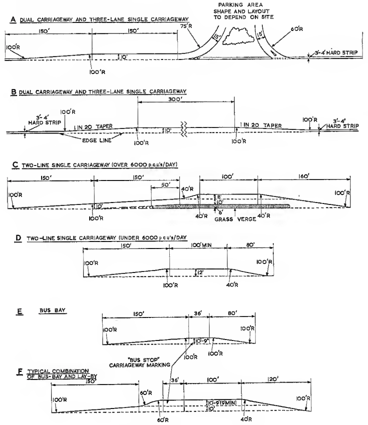


Fig. 3-16 Parking areas, lay-bys and bus bays

3.17 Safety Fences

Safety fences are needed where it would be particularly hazardous for vehicles to leave the road. This condition would obtain where there are roadside obstacles and structures such as bridge piers, or where there are narrow central reserves. They are necessary on both sides of roadways on embankment, where the height is 20 ft. or more, or where there is a road, railway or river at the foot of the slope. Where embankments are between 10 ft. and 20 ft. in height with no special hazards below, safety fences are normally required on the outside of curves less than about 2,800 ft. radius.

Safety fences should be designed to redirect a colliding vehicle without overturning, and to minimise the risk of rebounding with obvious danger to following traffic. Where possible the forces experienced by the vehicle occupants should be contained within tolerable limits. This requirement is largely a function of the lateral distance behind the fence available for safe vehicle travel, viz. a stiff fence is required where there is little or no space behind it.

Where there is little lateral distance behind the fence, safety fences should be mounted on strong posts with offset brackets to fend the colliding vehicle off the posts. For other conditions, tensioned beams carried on frangible posts are now available: these are effective in controlling the rebound of vehicles and can be easily replaced. The present minimum length for the tensioned beam type is 150 ft.

It is important that the ends of safety fences should be carried into the ground or turned away from the line of approaching traffic.

3.18 Street furniture, etc.

It is important to ensure that all obstructions such as telegraph poles, lamp posts, telephone kiosks, traffic signs and trees should be located so as to minimise dangers to vehicles overrunning the verge and to avoid obstructing visibility at junctions or on bends. Regard should also be had to the design of street furniture to minimise the danger from vehicles striking them. The protection of street furniture by safety fences is expensive and not wholly effective, and every effort should be made to avoid the need for this by proper siting. Obstructions other than signs and lamp posts are best sited at the back of the verge and at least 5 ft. from the carriageway. Traffic signs should be sited nearer the carriageway but preferably not closer than 4 ft. therefrom; lamp posts and posts for traffic signs should not normally be sited nearer the kerb than 5 ft.

For the siting of large road signs, extra highway width may be needed, both for the signs and the necessary visibility splays to them. Police boxes and telephone kiosks should normally be suitably sited at lay-bys.

Capacity and Geometric Design

3.20 Capacity and Speed

Design capacities are given in Design Table II, Column 3.

Where the standards of lane width, side clearance, sight distance and gradient laid down in this Memorandum are not provided, these capacities will be reduced. The aim should be towards achieving for as long lengths as possible the standards recommended. The capacities are given in passenger car units (rural standards) see Table 1–42. For roads with traffic lanes of less width than 12 feet and/or where the side clearance is generally restricted along the road, an estimate of the capacity may be made by reference to Table 3–20. For two-lane roads where the minimum overtaking sight distance is not provided throughout the length of road see Table 3–23 and for capacities on hills see Table 3–28.

The daily capacities in Design Table II given have been arrived at by taking the maximum hourly capacities and multiplying by 10. In the case of dual carriageways it was assumed that one carriageway carries 60% of the total flow in both directions. The maximum hourly capacities used in column 3 are given below.

24 ft. single carriageway	900 p.c.u's per hour total for both directions.
33 ft. single carriageway	1500 p.c.u's per hour total for both directions.
Dual carriageways	1000 p.c.u's per hour per lane in the direction of heavier traffic.

Where more detailed data exist on the peak hour or directional traffic proportions, the design should be based on these data; this may give a different daily capacity.

As traffic flow varies substantially hourly, daily and seasonally, the peak hour capacities as determined above are likely to be exceeded only at some week ends in summer months; at these times the increased traffic flows will result in reduced journey speeds.

Where future traffic is expected to exceed by a small margin (say up to 20%) the maximum capacity recommended for a particular road width, it will often be desirable to accept the road of lesser width and some overloading, particularly where the cost of providing a wider road is above average.

It should be noted that these capacities are for design purposes and may in practice be exceeded substantially, but under such conditions overtaking would be restricted and speeds lowered; for instance, a substantial overload may be acceptable as a transition stage between a two-lane road and the provision of a dual two-lane road, instead of providing a three-lane road which may have limited life. In some circumstances it will be satisfactory to allow higher traffic flows than given above, e.g. on long bridges or tunnels, on roads of a few miles in length carrying a high proportion of commuter traffic, or on roads where the ratio of August flow to average flow is abnormally high. On limited access dual carriageways a capacity of 1200–1500 p.c.u's per hour per lane may be adopted for conditions where reduced speeds at peak loads are acceptable. The flows given above may not be achieved if the capacities of the intersections are lower.

3.21 Design Speeds

The standards of design, superelevation, visibility, etc., should be correlated for any particular road at an appropriate design speed. The standards of design corresponding to different design speeds suitable for different widths of road are given in Design Table II. Design speeds below 60 m.p.h. are not recommended for Trunk or Principal Roads in rural areas. Where these standards are not economically practicable along particular lengths of a road, points of hazard should be indicated by appropriate signs; the general aim should, however, be to obtain comparable standards throughout as long a length of road as possible. For difficult country and for lightly trafficked roads it will seldom be economical to adopt the full standards recommended.

3.22 Minimum Stopping Sight Distances

Column 4 of Design Table II gives minimum stopping distances for different design speeds and classes of road. These sight distances should be measured between points 3 ft. 6 ins. above the centre of the lane on the inside of the bend. They should always be provided on dual and single carriageways.

3.23 Minimum Overtaking Sight Distances

On single carriageways sufficient visibility for overtaking should be provided on as much of the road as possible, especially where the traffic flows are expected to approach the design capacities given in column 3 of Design Table II. These are measured between points 3 ft. 6 ins. above the centre line of the carriageway.

Where minimum overtaking sight distances are not provided throughout the length of two-lane roads, Table 3–23 shows the effect on design capacity and speed.

Design Table II

Carriage-way Width in Feet	Design speed m.p.h.	Design capacity in p.c.u's/Day Rural Standards	Minimum Stopping Sight Distance	Minimum Overtaking Sight Distance	Minimum Desirable Radius	Absolute Minimum Radius with Maximum Super-elevation	Minimum Radius for Curves without Transitions	Stopping Sight Distance K Values for Crests	Stopping Sight Distances K Values for Sags	Overtaking Sight Distances K Values for Crests on Single Carriage-ways
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Dual 48 ft.	70	66,000 *	950	—	2,800	1,500	5,000	300	250	—
Dual 36 ft.	70	50,000 *	950	—	2,800	1,500	5,000	300	250	—
Dual 24 ft.	70	33,000 *	950	—	2,800	1,500	5,000	300	250	—
33 ft	60	15,000	650	1,400	2,100	1,100	4,500	150	150	700
24 ft	60	9,000	650	1,400	2,100	1,100	4,500	150	150	700
24 ft	50	9,000	425	1,200	1,450	750	4,000	65	100	500
—	40	—	300	950	900	500	2,000	35	50	300

*See Section 3.20.

Table 3–20* Capacities of roads restricted in width and/or clearance as percentages of the standard capacities

Type of Road	Carriage-way Width in feet	Obstruction one side of road Distance from Carriageway in feet				Obstruction on both sides of road Distance from Carriageway in feet			
		0	2	4	6 or more	0	2	4	6 or more
Dual 3 lane road	36	94	97	99	100	91	96	98	100
	33	91	93	95	96	87	92	94	96
	30	85	87	88	89	81	85	87	89
	27	74	76	77	78	70	75	77	78
Dual 2 lane road	24	90	97	99	100	81	94	98	100
	22	87	94	96	97	79	91	95	97
	20	82	88	90	91	74	86	89	91
	18	73	79	80	81	66	76	79	81
2 lane single carriageway road	24	85	91	96	100	70	81	92	100
	22	73	78	83	86	60	70	79	86
	20	66	70	74	77	54	63	71	77
	18	60	64	68	70	49	57	65	70

Note:

This table is not for the design of new roads but is included for the purpose of estimating traffic capacities on existing roads or of roads during maintenance works. The reduced capacities arising from obstruction at the roadside will not result from isolated obstructions such as road signs or where the length of road obstructed is short. Data is not available for 3 lane single carriageway roads, but it is reasonable to assume that the best estimate lies between the values for dual 2 lane and 2 lane single carriageway roads.

*Extract from Highway Capacity Manual (Special Report 87) of American Highway Research Board, Washington, D.C.

Table 3–23 Effect on design capacity and speed where minimum overtaking sight distances are not provided

Percentage of road length with substandard overtaking sight distance	0	20	40	60	80	100
Percentage of standard design capacity	100	90	80	65	50	30
Estimated reduction in average speed (in m.p.h.) of two-lane road carrying 900 p.c.u's/hr.	0	4	8	12	16	20

3.24 Horizontal Curvature and Superelevation

Curves should be laid out with the largest practicable radius. The superelevation should (1) normally balance out 40% of the centrifugal force, (2) not normally be steeper than 7% (1 in 14.5) except on existing roads or for loops at interchanges, or flatter than the normal crossfall of the road (see Section 3.29), and (3) be such that the residual sideways acceleration which must be balanced by the road friction does not exceed 0.15 g.

Fig. 3–24 shows the approximate superelevation for various design speeds and curvature. Where practicable, steep crossfalls should be avoided, and should normally be less than that corresponding to Desirable Minimum Radii.

The Absolute Minimum Radii are also indicated; such radii are undesirable and should only be used in exceptional circumstances. The Desirable Minimum and Absolute Minimum Radii are given in columns 6 and 7 of Design Table II.

For curves where the residual sideways force which must be taken by the road surface exceeds 0.10 g, care must be taken to maintain non-skid road surfaces and good drainage; there is otherwise a danger of skidding when unfavourable circumstances obtain, e.g. worn tyres, wet or greasy road surfaces, vehicle speeds in excess of design speed, etc.

In cases where it is not immediately practicable to provide proper curvature and superelevation, and where accidents due to skidding occur, steps should be taken to resurface with material of high skid resistance.

The superelevation of short lengths of curves, even of long radius, required for angles of small deviation is often unsatisfactory, and in such circumstances it is necessary to vary the road alignment to provide sufficiently long curves for superelevation to be applied.

On curves which do not require superelevation, adverse camber should always be replaced by favourable crossfall up to radii 4 times those given in column 6 of Design Table II. On long curves of greater radii, it may also be desirable on aesthetic grounds to eliminate adverse camber.

Superelevation and removal of adverse camber should preferably take place throughout the length of the transition curve, but for aesthetic or other reasons it may be necessary for part of the superelevation to be applied outside the transition length. On curves of large radius where no transition curves are needed the superelevation should be similarly applied on an assumed transitional length.

Various assumptions may be made which result in determining the assumed transitional length (or superelevation run-off) in different ways but as a simple rule the difference in grade between the inner edge of the carriageway, and the outer edge should not exceed 1.0%.

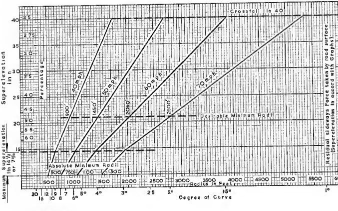


Fig. 3–24 Superelevation of curves

3.25 Transition Curves

Transition curves should be provided on all curves the radius of which is less than those given in column 8 of Design Table II.

3.26 Widening on Curves

Widening on curves is required for substandard conditions as follows:

(a) For roads with 12 ft. traffic lanes where the radius is less than 500 ft. the width should be increased by 1 ft. per traffic lane.

(b) For roads with traffic lanes of less than 12 ft., widening should be as follows:
radius between 300 and 500 ft.—widening 2 ft. per traffic lane, up to a maximum of 13 ft.
radius between 500 and 1,000 ft.—1½ ft. per traffic lane up to a maximum of 12 ft.
radius between 1,000 and 1,500 ft.—1 ft. per traffic lane up to a maximum of 12 ft.

The widening should be effected by increasing the width at an approximately uniform rate along the transition curve. In the improvement of existing curves the widening should generally be made on the inside of curves. Roads marked as two lane where each lane exceeds 12 ft. do not require widening on curves.

3.27 Vertical Curves

At all changes in gradient vertical curves should be provided. The curvature should be large enough to provide sight distances which allow for safe stopping at design speed. For crests, the stopping sight-distance should be measured from points 3 ft. 6 ins. above the carriageway; for sags the stopping sight-distance is that required for headlamp beams to show up objects on the carriageway. On single carriageway roads there should, where practicable, be sufficient visibility distance on crests to allow for overtaking. The stopping sight distances and overtaking sight distances for different design speeds are given in columns 4 and 5 of Design Table II. The minimum curve lengths referred to can conveniently be determined by multiplying the K values given in Columns 9, 10 and 11 of Design Table II by the algebraic difference in grades expressed as percentage, e.g. +3% grade to −2% grade indicates a grade change of 5% so that for a design speed of 70 m.p.h. the length of vertical curve on a crest could be 1,500 ft. and on a sag 1,250 ft.

Where for any reason the standards quoted are not yet attainable they may be reduced, but never to less than 75% of the values obtained by using the design table.

It should also be checked that the minimum length of curve is never less than 3 times the design speed in m.p.h., e.g. where the design speed is 60 m.p.h. the minimum length for any change in grade should be 180 ft.

3.28 Gradients

A gradient of 1 in 25 (4% grade) should normally be regarded as a desirable maximum, though in hilly country steeper gradients may have to be adopted, particularly on the less important routes: some relaxation is applied to slip roads, see Section 4.646.

Crawler lanes for ascending traffic are recommended where the gradient, length of gradient, traffic volume and proportion of heavy traffic warrant them. Table 3–28 sets out for different conditions the road capacity on hills as a percentage of that on the level*. It is recommended, however, that for hills an overload must be accepted, and crawler lanes should only be provided where the predicted future traffic is greater than the capacity given in Design Table II (or as modified by Table 3–20) by 100% in the case of two-lane roads and by 50% in the case of dual carriageways.

*Derived from data in Highway Capacity Manual (Special Report 87) of American Highway Research Board, Washington, D.C. 8

Example: Dual 2 lane road is on 2 mile hill at 4% grade, p.c.u./vehicle ratio = 1.3. Normal capacity = 33000 p.c.u's/day. From Table 3–28 capacity on hill = .52 × 33000. With recommended overload of 50%, the maximum acceptable capacity = .52 × 33000 × 1.5 = 25800 p.c.u's/day. If the predicted future traffic exceeds this, a crawler lane will be desirable. For three lane roads where the length exceeds the critical length given for the gradient in the table below, one of the lanes may be marked as a crawler lane by means of offset double white lines.

Gradient (%)	3	4	5	6	7
Critical Length (ft.)	1600	1100	800	680	550

For all road widths the full width of the additional lane should be provided at a distance from the bottom of the hill (i.e. about the centre part of the vertical curve) of about half the critical length of gradient given above: this should be preceded by a taper of about 200 feet. This crawler lane should end at about 500 feet beyond the summit followed by a taper of about 200 feet.

Table 3–28 may also be used as a guide in choosing vertical alignment so that gradients and lengths are kept below the critical values.

3.29 Road Camber and Crossfall

On straight sections of road the crossfall from the centre of single carriageways or from the central reserve of dual carriageways to each side should normally be straight and at a fall of about 1 in 40. At junctions the normal camber of the major road should be retained across the junction and the side road graded into the channel line of the major road.

Where dual carriageways are on long bridges or viaducts a crossfall towards the centre is acceptable where required for drainage or structural reasons.

3.30 Vertical Clearance

The standard minimum headroom for bridges over a road is 16 ft. 6 ins: this should be maintained over the carriageway and over any hard shoulders or hardened strips where they are provided. Where future maintenance of the carriageway is likely to lead to a raising of the carriageway level, not more than 3 ins. additional clearance may be provided initially. Where the existing headroom exceeds the standard and a reduction would affect local industry, greater clearance may exceptionally be justified. Exceptionally, it may be found acceptable to provide headroom as low as 15 ft. if this can be done without detriment to reasonable traffic requirements and with a worthwhile saving in construction costs.

3.31 Pedestrian Bridges and Crossing Facilities

Facilities to assist the crossing of roads by pedestrians are not usually needed except on important roads through villages. For roads restricted to 30 or 40 m.p.h. the measures appropriate to urban areas apply (See Urban Traffic Engineering Techniques H.M.S.O. 1965[8], and Roads in Urban Areas[9]). For other roads the alternatives to consider are dualling (or provision of refuges) so that pedestrians can cross the road in two stages—or the provision of pedestrian bridges or subways. The provision of refuges or dual carriageways will not usually be found necessary where there are light concentrations of pedestrian traffic on single carriageway roads with volumes less than about 9000 p.c.u's a day. For dual carriageway roads carrying over 15000 p.c.u's a day subways or bridges may be needed if pedestrian flows are sufficiently concentrated.

New roads should be located so as to minimise the need for pedestrian facilities, by by-passing areas of development.

Table 3–28 Carriageway capacities on hills expressed as percentages of those on level

Gradient %	Predicted Future P.c.u./Vehicle Ratio*	2 LANE ROADS: Length of Gradient in Miles							DUAL 2 LANE CARRIAGEWAYS: Length of Gradient in Miles						Gradient %	Predicted Future P.c.u./Vehicle Ratio*
		¼	⅜	½	¾	1	1½	2	¼	½	¾	1	1½	2		
3	1.4	80	62	50	38	33	30	28	100	86	78	70	67	64	3	1.4
	1.3	83	67	56	44	38	35	33	89	89	81	81	71	63		1.3
	1.2	87	73	63	53	47	43	40	86	86	80	80	75	70		1.2
	1.1	91	83	77	67	61	58	56	81	81	81	81	80	78		1.1
4	1.4	64	44	34	26	23	21	20	100	78	64	59	52	46	4	1.4
	1.3	68	54	40	32	27	26	24	89	81	68	63	56	52		1.3
	1.2	74	58	49	39	34	33	31	86	86	75	70	64	59		1.2
	1.1	83	71	62	54	49	47	44	78	78	78	76	75	73		1.1
5	1.4	50	33	24	20	18	17	16	100	64	59	50	45	39	5	1.4
	1.3	55	38	29	25	22	21	20	89	68	63	55	50	44		1.3
	1.2	60	45	36	32	29	27	26	80	79	68	62	57	52		1.2
	1.1	75	61	51	45	43	41	39	76	73	73	71	69	67		1.1
6	1.4	37	25	18	17	16	15	14	100	59	50	46	41	37	6	1.4
	1.3	44	30	22	19	18	17	17	89	63	55	49	44	40		1.3
	1.2	53	37	28	25	24	23	23	80	70	62	56	51	48		1.2
	1.1	64	52	43	38	37	35	34	76	73	71	69	67	65		1.1
7	1.4	25	18	15	14	13	12	11	100	59	50	45	39	34	7	1.4
	1.3	29	22	17	15	14	14	14	91	63	56	50	42	39		1.3
	1.2	36	28	23	20	19	18	18	77	69	62	57	50	48		1.2
	1.1	51	43	35	32	31	29	29	76	71	69	67	65	62		1.1

*It is important to note that the P.c.u./Vehicle Ratio to be used is that expected for the Design year—see Memorandum on Traffic Prediction for Rural Roads.

3.40 Bridges (*see also Section 2.2*)

Layout

On bridges the edge of the carriageway over the bridge should be indicated by a 4 in. high kerb (normally of the type used on the bridge approaches) so that there is never less than 4ft. between the parapet and the edge of the carriageway.

On long bridges or viaducts (unless separate structures are provided), economic considerations make it necessary to reduce the width of a central reserve (see Section 3.15). In order not to spoil the flowing alignment of the road any alteration in width should be carried out gradually along curves on the approaches to the bridge.

Where dual carriageways are carried on separate structures for each carriageway the verge on the offside of each carriageway should not be less than 4 ft. If the gap between the bridges is spanned by a grid or deck it should be designed to withstand H.A. loading as specified in British Standard 153[8]. Where the space between the bridges is not covered the offside of each carriageway should be protected with appropriately strengthened parapets as for the nearside of the road.

Grassed surfaces should not be continued over bridges; verges and central reserves should be suitably paved.

Clearances

The clearance between the nearside edge of a carriageway and the face of an abutment or pier should not be less than 4 ft.

Bridge piers on a central reserve should be protected by safety fences which should be at least 4 ft. from the edge of the carriageway. Where possible a clearance of at least 12 in. should be provided between the back of the fence supports and the bridge pier. A minimum headroom of 16 ft. 6 ins. must be provided and maintained over carriageways and hard strips.

Bridges over railways should be designed to minimise costly interference with train services during construction. The provision of lateral clearances greater than those stipulated may prove economical if the slowing of trains is thereby avoided.

Ministry of Transport Publications relating to the design of bridges are given under References on page 44, Nos. 19–38.

3.41 Traffic Signs and Carriageway Markings

An efficient system of traffic signs and carriageway markings is essential for the safety and free flow of traffic.

Information on signs and carriageway markings is given in:

Informatory Signs for use on all-purpose Roads (H.M.S.O. 1964)[10]

The Traffic Signs Regulations and General Directions 1964 (H.M.S.O. 1964)[21]

Traffic Signs Manual (H.M.S.O. 1965)[4]

The Traffic Signs General Directions (H.M.S.O. 1966)[22]

The Traffic Signs (Amendments) Regulations (H.M.S.O. 1966)[23]

British Standard No. 873[24]

Traffic signs should be frequently reviewed and special attention given to sites where accident studies suggest the need for improvement.

3.42 Lighting of Roads and Junctions

The lighting of rural roads and particularly of important junctions is effective in reducing accidents: from some studies a 30% reduction in personal injury accidents occurring at night appears likely. The extent to which lighting should be applied to rural roads and the standards of lighting are under investigation and experiment. The results should provide an economic basis for decision.

The technical standards for road lighting, which include the siting of lighting columns, are given in the British Standard Code of Practice C.P. 1004: 1963,[25] or subsequent revision, issued by the British Standards Institution. The quality of design within the range of Group A should be related to the traffic and road conditions. For two-lane roads a good standard of Group B lighting may be sufficient.

The lighting of roundabouts and important intersections (particularly those which employ a number of channelised features) is needed to ensure the early recognition by drivers of the obstruction and gives them adequate visibility for following the appropriate paths and for merging, cutting, etc., with other traffic.

3.43 Roadside Advertisements

Information on the safety aspects of the control of roadside advertisements is given in Ministry of Housing and Local Government Circular No. 11/62 Town and Country Planning (Control of Advertisements) Regulations 1960—Public Safety (H.M.S.O. 1962)[16]. (For Scotland—Department of Health for Scotland Circular No. 57/61, Town and Country Planning Act, 1947—General Guidance and Public Safety;[17] and S.I. 1961 No. 195—Town and Country Planning (Control of Advertisements) (Scotland) Regulations, 1961[18]).

3.44 Access

Unlike the motorways, all-purpose rural roads have to provide access to land and property. Future development can be controlled but a major problem is that of providing safe access to existing development, farms and land.

It is important that at a gated access the gates should open inwards and that sufficient space should be provided off the carriageway to accommodate a standing vehicle, and if necessary the gate should be set back to allow for this. Adequate entry and exit radii and visibility should also be provided (see Sections 4.30 and 4.34); the recommended minimum radius is 20 ft. and the minimum verge width of 6 ft. will normally give adequate visibility if clear of obstructions.

One of the problems affecting rural roads arises from the movement of farm animals; this may be twice daily for milking herds, or by periodic movements for changes of grazing. Some of the methods employed to meet this problem are land exchange, the provision of internal farm roads, resiting of farm buildings, use of milking bails, change in method of farming, etc; many of these are to be preferred to the provision of cattle creeps or tunnels, the expenditure on which may often prove unnecessary in the long term, due to changes in farming methods.

The movement of cattle along a road can best be catered for by driftways within the farm or in some cases by the widening of roadside verges, which may need to be hardened and drained.

The recommended dimensions of tunnels for cattle are 18 ft. wide by 8 ft. high and for farm implements 15 ft. wide by 14 ft. high; the recommended width between parapets for accommodation bridges is 15 ft.; the design loading should normally be ½ HA. Approach gradients should not exceed 1 in 10.

On dual-carriageway roads breaks in the central reserve to provide access should, for safety reasons, be kept to a minimum and confined where possible to the more important road junctions. The problem of access can best be overcome by special provision at junctions for U turns and right turns (see examples Fig 4-52(6) and (7)). Where the spacing between junctions is too great and would result in an unreasonable detour, a U turn may be provided for by a deceleration lane in the central reserve and local widening on the far side of the road for the turning vehicle: suitable dimensions can be obtained from the section on junction design.

4 The Road Junction

4.10 General

Road junctions are a major source of danger and, when over loaded, of traffic congestion: particular care is needed to ensure that the type of design selected and the layout are best suited to traffic needs. On rural roads most of the junction accidents occur at the very numerous lightly trafficked junctions, and from the safety aspect as much attention is needed to these lightly trafficked junctions as those where heavier conflicting traffic movements occur. For the more important junctions estimates of future traffic and its distribution at peak hours are needed before steps can be taken for their design or modification. In general, peak hour volumes should be determined in the way outlined in Section 1.41, taking also into account other factors which may influence the traffic pattern, e.g. new development of land or the effect of new motorways or other major roads.

In some instances traffic information may be limited to the total daily flows and peak hourly flows may be assumed to be 10% of the daily flow allocated in the ratio of 60:40 directionally. This method will not be satisfactory where the proportion of turning traffic is heavy.

4.11 Influence of speed on choice of type of junction

There will be a need on rural roads for a large number of minor road junctions which give access to the rural area as well as the more important junctions linking the through routes. The minor junctions are best catered for by suitably designed 'T' or staggered junctions whilst the choice of design for major junctions may be for uncontrolled, channelised, roundabout, grade separated operation or in exceptional cases signal-controlled operation. Whilst grade separation is needed at important junctions there will be a need for less costly types at less important ones. For junctions along fast stretches of road it is desirable to avoid as far as possible the use of types of junctions which necessitate abrupt changes in speed on the main road, and roundabouts, and particularly traffic signal-controlled junctions, have this limitation.

Where grade separation is not justified, the wider use of channelised junctions is recommended. Roundabouts, however, form a useful transition stage before applying grade separation to dual-carriageway roads, and are appropriate as a permanent measure for single-carriageway roads, particularly when crossing flows are similar in volume. Roundabouts are often suitable at junctions where traffic disperses amongst several roads.

There should be consistency in the types of junctions used as far as is reasonable; in particular a mixture of grade separated junctions and roundabouts along the same road should be avoided. On fast important roads where grade separation is justified at the major junctions it may also be advisable to consider the case for grade separation at some less important ones, and to permit only left turns at others. On such roads the number of minor junctions should be kept to a minimum by rearrangement of the road pattern.

4.12 Influence of traffic volumes on choice of types of junctions

The operation of uncontrolled junctions depends on the frequency of gaps naturally occurring between vehicles in the main road flow which are of sufficient duration to permit vehicles from the side road to cross it, and in consequence such junctions are limited in capacity: by the application of channelisation methods to obtain the maximum capacity the use of uncontrolled junctions can be extended to cover a wide range. The capacity of uncontrolled junctions is discussed in Section 4.40. Higher capacities can generally be obtained by roundabouts and although no definite limit has been demonstrated it is usual to regard a weaving flow of 3,000 p.c.u's/hour on one side of the roundabout as a practical upper limit, although grade separation should be considered at lesser flows than this.

Delay is also an important factor and the saving in time otherwise lost by delay will sometimes justify grade separation even when the flow is somewhat (say 10%) below the capacity of an uncontrolled junction designed to the maximum standards of this memorandum. The economic rate of return will usually be low at these capacities but grade separation may be acceptable on account of the speed characteristics of the route or where the future volumes necessitate grade separation. The best economic return may often be obtained by phased construction, e.g. a roundabout constructed first and later a flyover, and such a possibility should always be explored.

Principles of Design

4.20 Safety

The general principles of design given in this section take account of such considerations as safety, operational comfort and capacity.

It is frequently desirable to consider the effect of the design of existing junctions from the safety aspect. This is best done by studying the frequency with which types of accidents occur so that appropriate measures may be decided: for example, if accidents involving vehicles emerging from a side road are frequent, attention to visibility splays, carriageway markings, or traffic islands is often indicated.

It is recommended that a systematic record of accidents should be kept as indicated in Ministry of Transport Circular No. 734 and in paras. 180–189 of the Memorandum on Urban Traffic Engineering Techniques.[8]

It can also be useful for deciding priorities to compare the frequency of accidents occurring at different junctions in relation to traffic volumes, and it has been found that for comparable designs the number of accidents A occurring will be approximately equal to $C\sqrt{Qq}$ where Q and q are the traffic volumes on the main and side roads respectively and C is a value depending upon the design of the junction and traffic pattern. Thus $C = \frac{A}{\sqrt{Qq}}$ and A can, for example, be the number of personal injury accidents in a year, and Q and q can conveniently be measured in thousands of vehicles per day (total for both directions of flow for each road). For example, at 3-way junctions the C values for the right turn into the side road or right turn from the side road can be determined from the volumes and accidents occurring. Junctions with high C values should be considered for priority treatment.

The best information available on the risks involved in different traffic movements for simple 'T' junctions is given below, and the percentages may be used to indicate where best returns in accident reduction can be achieved:

Accidents between a through vehicle and one

(a) turning right from the main road	37%	75%
(b) turning right from the side road	25%	
(c) turning left from the main road	3%	
(d) turning left from the side road	10%	
Two turning vehicles	12%	25%
One vehicle accident	13%	

The percentages relate to the junctions studied and will vary widely according to design and traffic distribution. It is not yet possible to estimate precisely the reduction in accidents likely to follow changes in design but, for example, in some studies the improvement of sight lines effected an overall reduction of 30% in accidents.

A study in California yielded some similar information to that above. It concerned accidents at intersections on divided highways and found that the accidents per year $N = 0.000783\ V^{[illegible]}S^{[illegible]}$ where V was the average daily flow entering the junction from the divided highway and S that from the side road. About 60% of the accidents were non-injury.

4.21 Minimising the number of junctions

From the previous section it will be clear that a considerable saving in accidents is therefore likely to result by eliminating lightly trafficked side road connections on to main roads. For example, where two minor roads can be connected together before joining a main road the accidents should be reduced by about 30%.

4.22 Constraining drivers from hazardous movements

Drivers often take chances such as heedlessly crossing a major road. Prevention of such movements can reduce accidents; for example, by converting cross-roads into properly designed staggered junctions, the accidents have been reduced on average by about 60%.

Guide islands can be used to similar effect, e.g. for constraining drivers from taking a right turn on a bend until a point of adequate visibility is reached.

Guide islands can also be used for slowing down drivers, or for guiding them into suitable positions for cutting or merging with other traffic streams and for warning drivers that they are approaching a major road.

4.23 Driver comprehension

It is important that a driver on entering a junction should be able to discern quickly either from the layout or from traffic signs the path he should follow and the actions of intersecting or merging vehicles. To give effect to this, the layout, traffic islands, signs and carriageway markings should be used to define the paths to be taken: uphill approaches to a junction make it difficult for drivers to comprehend the layout.

Adequate visibility splays should be provided to enable drivers to make appropriate decisions in sufficient time to eliminate possible accident risks.

4.24 Natural traffic movements

As far as is consistent with other considerations, the layout should be designed so as to follow the natural vehicular paths: this improves the smoothness of operation of a junction and makes it readily understood by drivers. It follows, for example, that unduly sharp radii or complex paths involving several changes in direction shall be avoided if practicable.

4.25 Designing to meet traffic pattern

The layout should be designed to suit the traffic pattern, e.g. principal movements are given to the easiest paths in separate channels, curves are provided for slowing down minor streams into stopping positions, and the alignment and dimensions of slip roads, gaps in traffic islands, etc., are suited to the various traffic movements.

4.26 Separation of traffic conflicts

Some of the many cutting, merging or diverging movements can be usefully separated so that the number of traffic movements at any point is reduced. This separation of conflict results in greater safety as drivers are then only faced at any one time with simple decisions as to choice of movements: it also results in greater capacity.

The extent to which traffic conflicts should be separated depends on the traffic volumes in conflict and this is discussed in Section 4.42.

4.27 Provision of waiting areas for vehicles

Where traffic has to wait in order to cross a traffic stream, such as when making a right turn from a major road, a safe waiting space (right turn lane) should be provided in the middle of the road large enough to accommodate the likely number of vehicles waiting to turn: the importance of providing in this way for right turning traffic is borne out by the very large accident rate of vehicles involved in this movement, where no such space has been provided.

4.28 Merging and diverging

Where traffic leaves the main road on the near side the layout should preferably permit the divergence of the two streams at a small angle and approximately equal speeds. It is also advantageous for left-turning merging movements to take place at a small angle so that vehicles joining the main stream may do so at the speed of traffic in that stream: this does not apply to junctions without acceleration lanes where vehicles on the minor road may be expected to stop before entering gaps in the major stream.

4.29 Size of junction

Junction size affects both capacity and operational characteristics. If the separation of vehicle conflict points is to be effective the dimensions of the junction must be large enough to ensure that a driver is able to distinguish in adequate time between those vehicles which will conflict with his intended path and those which will not; only in this way can the traffic gaps be used effectively. Layouts which have numerous small traffic islands should if possible be avoided as these may be confusing and ineffective.

Geometric Standards

4.30 Visibility Distance

At uncontrolled junctions visibility splays should be provided so that a driver approaching from a minor road can have unobstructed visibility to the left and right along the main road so that he may judge when an adequate gap occurs in the traffic flow for his vehicle to turn into the main road. The visibility should be obtainable between points 3 ft. 6 ins. above the road level over the area defined by:

(a) a line 36 ft. long along the centre line of the minor road from the continuation of the line of the nearer edge of the carriageway of the major road. For culs-de-sac, or very lightly trafficked roads, the 36 ft. dimension may be reduced to 20 ft. in difficult circumstances; this relaxation should not normally be applied where the minor road is a classified road.

(b) a line of the lengths given in Column 2 of Design Table III measured along the nearer edge of the major road carriageway from its intersection with the centre line of the minor road.

(c) a straight line joining the terminations of the above lines.

Standards for roads in urban areas are given in Roads in Urban Areas (H.M.S.O. 1965).[5]

Where the main road has dual carriageways, with a central reserve of adequate width to shelter traffic, the visibility splay to the left is not essential, but the central reserve should be clear of obstructions for the length given in Design Table III, Column 2. It is advantageous on capacity grounds to increase where practicable the distances (b) referred to by up to about 50 %: this allows several vehicles to emerge when large gaps in traffic occur. It is not advisable to increase the distance (a) to substantially more than 50 ft. except at right-hand splay junctions (see Section 4.52(3)).

At the approach to a roundabout there should be unobstructed visibility to the right of at least 200 ft. along the preceding weaving section from a point at least 50 ft. (preferably 75 ft.) back from the nearside of the carriageway of the roundabout. The forward visibility from points on the centre of the carriageway of the roundabout should not be less than the weaving lengths ahead or 200 ft. whichever is the less.

Some essential traffic signs will need to be erected within the visibility areas referred to, but great care should be taken to minimise their obstructive effect.

4.31 Speed-Change Lanes

The value of acceleration and deceleration lanes depends upon the speed of traffic, the volume of traffic on the major road and on the volume of traffic entering or leaving the side road. Nearside speed change lanes are recommended to be uniformly tapered, with a maximum set back of 18 ft. at the tangent point of the curve leading into or out of the minor road. The turning lane should be reduced in width to 14 ft. by carriageway markings (See fig. 4.51). For widths of connecting slip roads see Section 4.35.

4.32 Acceleration lanes

An acceleration lane should be designed so that vehicles turning left from the minor road may join the traffic flow on the major road at approximately the same speed as that of the nearside lane traffic in the major road; acceleration lanes also improve capacity by enabling the use of short traffic gaps and by providing storage space for traffic waiting to merge when large traffic gaps occur. Acceleration lanes are recommended for roads referenced 1–2 in Design Table I where the future traffic on the acceleration lane is expected to be more than 1,000 p.c.u's day: they should, however, usually be provided at grade separated intersections.

Recommended lengths of acceleration lanes for different main road design speeds are given in Column 3 of Design Table III and a typical layout is given in Fig. 4–51(4). In difficult conditions, sub-standard lengths may have to be accepted, but they should never be less than half those recommended.

Where acceleration lanes are on a down gradient their length may be reduced to 1—0.08G times the normal length, where G is the down gradient expressed as a percentage. For acceleration lanes on an up gradient the length should be increased to 1 + 0.12G times the normal length, where G is the up gradient expressed as a percentage.

4.33 Deceleration lanes

Deceleration lanes are of greater value than acceleration lanes because the driver of a vehicle leaving the highway has no choice but to slow down any following vehicle on the through lane if a deceleration lane is not provided. Deceleration lanes are needed on the nearside for left turning traffic and onto the right turn lane where provision made is for right turning traffic.

The length of nearside deceleration lanes should be sufficient for vehicles to slow down from the average speed of traffic in the nearside lane to the speed necessary for negotiating the curve at the end of it: in order to make deceleration lanes effective the curve radius must permit a speed of at least 20 to 25 m.p.h. (not less than 100 ft.). Recommended lengths of nearside deceleration lanes are given in Column (4) of Design Table III and a suitable layout is given in Fig. 4–51(3). Nearside deceleration lanes are recommended for junctions on roads referenced 1–2 in Design Table I where the future traffic on the deceleration lane is expected to be more than 750 p.c.u's day; they should, however, always be provided at grade separated intersections.

Where the number of traffic lanes on a road is reduced immediately beyond a slip road, in order to avoid entrapping through vehicles in the slip road the carriageway should be constructed to full width to the exit nose and a taper length of 600 ft. provided beyond it.

Right-turn deceleration lanes in the central reserve should be provided at all gaps for right-turning traffic on dual-carriageway roads. On three-lane roads the centre lane should be marked for right turning traffic where the product of estimated future cutting flows in p.c.u's/day is more than one million. The widening of two-lane single-carriageway roads to provide right-turn deceleration lanes in the centre of the road should be considered at the

Design Table III

Design Speed m.p.h.	Visibility Distance at junctions (ft.)	†Acceleration Lane length including nose (ft.)	†Nearside Deceleration Lane length including nose (ft.)	Right turn Deceleration Lane length including taper (ft.)
1	2	3	4	5
70 (Dual carriageways)	700	1200	600	600
60	700	900	500	500
50	600	(700)	(450)	450
40	500	(500)	(400)	400

Note:

( ) Speed Change lanes not usually needed except in special circumstances, e.g. where grade separation is provided at a junction.

† See Sections 4.32 and 4 33 for flow criteria.

same levels of flow as for three-lane roads. These provisions may be made for lesser flows where accident records warrant them, or on two-lane roads where they can readily be incorporated in a realignment or other scheme. On overloaded three-lane roads or where the road junction is on a crest, it is usually desirable to construct short lengths of dual carriageways and provide right-turn deceleration lanes for right-turning traffic.

The lengths of right-turn deceleration lanes should be sufficient for vehicles to slow down to a stop from the average speed of vehicles in the off side lane: omission of these lanes will usually result in numerous head to tail collisions. Recommended lengths for right-turn deceleration lanes are given in Column 5 of Design Table III and a suitable layout is given in Fig. 4–51(4). These lanes should not be less than 10 ft. wide and parallel-sided with entry and return radii of 600 ft. giving a taper of 150 ft.

Even if it is not practicable to provide the full length of deceleration lane (right-turn or nearside) sub-standard lengths are still of great benefit but they should not be less than half the recommended lengths.

Where deceleration lanes are on an up gradient their length may be reduced to that obtained by multiplying the recommended length by $1-0.03G$ where G is the gradient expressed as a percentage. For deceleration lanes on a down gradient their length may be increased to that obtained by multiplying the recommended length by $1+0.06G$.

4.34 Radius of curves

The radius of a curve has a large influence on the speed of vehicles. At junctions, particularly the more complex ones, such as the channelised types or those involving grade separation, there is a need to select appropriate curve radii to influence vehicle speeds at various points. For example, an entry speed of 15 m.p.h. would be a suitable speed from which a driver can either stop or accelerate when entering a major road.

The speed at which most drivers follow a curve may be taken to be $2\sqrt{r}$ m.p.h. up to about 35 m.p.h. where r is the radius of the curve in feet. Design Sheet IV, columns (1) and (2) give the design speeds for different radii.

Recommended radii for left turns are as follows:

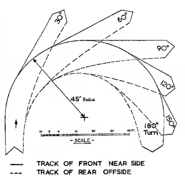
	Desirable Radii	*Minimum Radii*
Into major road without acceleration lane	60 ft.	35 ft.
Into major road with acceleration lane	100 ft. or more	75 ft.
From major road without deceleration lane	„ „ „ „	75 ft.
From major road with deceleration lane	„ „ „ „	75 ft.

It is usually advantageous to use compound curves.

For right-turning movements within a junction, lower radii are necessary. Fig. 4–34 shows suitable tracks for design purposes; templates to appropriate scales can be prepared from this diagram.

4.35 Widths of carriageways in junctions

The width of carriageways in junctions should be sufficient for the traffic flow plus an allowance for curvature and on long sections for stalled or broken-down vehicles. For freely running slip roads, such as simple left turns into or out of the minor road, the recommended width given in Design Table IV can be assumed to have a capacity up to 1,200 p.c.u's/hour. The widths in Column 3 are appropriate for short lengths of road, but for longer slip roads (say over 200 ft. in length) the widths given in Column 4 should be used. Widths for two-way traffic or two-lane roads are given in Column 5. Where the flow is interrupted, for example where vehicles make a right turn from the minor road, extra width may be needed to cater for two streams of vehicles (see Section 4.403).



Based on outer radius of 28 Ton(Gross) 8 Wheel Rigid Vehicle and inner radius for 32 Ton(Gross) 4 Axle Articulated Combination

Fig. 4–34 Turning diagram for maximum sized British vehicles

Design Table IV Widths of carriageways in junctions

Inner Radius ft.	Design Speed m.p.h.	Single lane width ft.	Single lane width with space to pass stationary vehicle ft.	Two lane width for one or two way traffic ft.
(1)	(2)	(3)	(4)*	(5)
35	12	18	34	38
50	15	17	31	35
75	18	16	28	32
100	20	15	26	30
125	22	14	25	29
150	25	14	24	28
200	28	14	23	27
300	35	14	23	27
400	38	14	22	26
500	40	14	21	25
Straight		14	20	24

*The excess width in Column (4) over Column (3) may be constructed to lower standards provided that the surface is either hatched or of a different surface to discourage its use by normal moving traffic.

Capacity of Junctions

4.40 Uncontrolled Movements

4.401 Merging (Fig. 4-40(a)). This movement may take place by means of an acceleration lane or simply by an entry curve.

As the volume of traffic from a slip road which may safely merge with the major road traffic depends on the volume on the major road, a detailed analysis of flows at each peak period is required. It is not sufficient to assume that the maximum possible flow of the two merging streams of traffic will equal the capacity of the major road beyond the junction: this is due to the screening effect of traffic on the nearside lane which prevents traffic from the slip road filling the traffic gaps in the offside or middle lanes. Consequently the slip road volume cannot exceed the capacity provided by the gaps in the nearside streams unless an extra lane is provided on the nearside of the major road.

There is some difficulty in determining the nearside lane volume as the distribution of traffic between lanes depends not only on the volume of traffic but also on the tendency of traffic to shift from the nearside lane in the vicinity of junctions. There is little experience in this country on which to make a close assessment of the traffic distribution near junctions, but from experience in the U.S.A. an estimate can be made of the maximum slip road flows for different main road flows. Fig. 4-401 shows these flows for different carriageway widths.

Where the expected slip road flows exceed the maximum flows given in the figure it will be necessary to provide an additional lane to the main road.

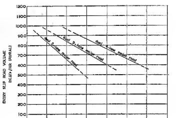


Fig. 4-401 Capacity of merging flows

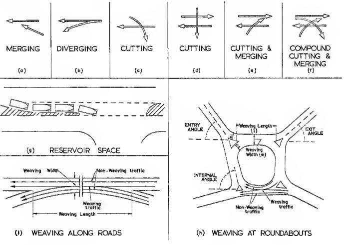


Fig. 4-40 Uncontrolled movements

The slip road capacities do not apply when there are other nearby road connections and their effect is to increase or decrease the volume in the nearside lane according to their position and direction of flow. The effect is most marked, from some studies, by on-road connections upstream within 500 ft., and somewhat less at 500 ft. to 1000 ft. distance.

For the effect of downstream off-road connections reference should be made to Sec. 4.409.

Where no acceleration lane is provided the capacity of the slip road entry onto a dual two-lane road may be assumed for design purposes to be 200 p.c.u's/hour less than with an acceleration lane; greater differences have sometimes been experienced.

4.402 Diverging (*Fig.* 4–40(*b*)). The capacity of a slip road exit may be taken to be 1,200 p.c.u's/hour provided the deceleration lane is well designed and sign-posting gives adequate warning well in advance of the junction. In the case of dual three or four-lane roads when the flow off the slip road approaches the capacity of one lane, it may be appropriate to reduce the main carriageway by one lane beyond the slip road connection.

4.403 Cutting (*Fig.* 4–40(*c*) & (*d*)). A cutting manoeuvre may be a simple right-turn cut of one stream of traffic across the opposing stream, or across a two way stream of traffic. These movements take place in the naturally-occurring gaps in the traffic streams which are of suitable duration; the larger traffic gaps are utilised by several vehicles cutting at one time.

As in general the flow of traffic on a road approximates to a random distribution, calculation can be made of 'capacity': this capacity depends on the size of gaps required and the volume and speed of the traffic stream which must be crossed. For conditions of good visibility the curves shown in Fig. 4–403 give the minor road flow which can cross different major road flows either one or two-way for different sizes of traffic gaps. For one-way flows, e.g. across one carriageway of a dual-carriageway road it would be reasonable to assume average minimum gaps of 4–6 seconds, and across two-way carriageways 6–8 seconds. For the higher design speeds the larger gaps should be used.

These capacities relate to a single lane from the minor road; capacity can be increased by providing two lanes if these could be used effectively.

4.404 Cutting and Merging (*Fig.* 4–40(*e*)). Unlike the simple cutting of two-way flows this manoeuvre requires traffic gaps or sufficient duration for the turning vehicle to accelerate to a suitable speed to join the far traffic stream and traffic gaps of 8–12 seconds may be needed; Fig. 4–403 gives the appropriate volumes.

4.405 Compound cutting and merging (*Fig.* 4–40(*f*)). This type of movement takes place at a simple 'T' junction. The normal sequence is for the right turn (Rt) into the minor road to precede the right turn (R_2) from the minor road. Precise calculations are complex and for most purposes it is sufficiently accurate to add half of R_1 to R_2 and from Fig. 4–403 the capacity can be calculated using the gap sizes appropriate for cutting and merging given in Section 4.404.

4.406 Reservoir Space (*Fig.* 4–40(*g*)). Where vehicles wait for opportunities to cross traffic streams, queues of vehicles develop and need to be accommodated in suitable 'reservoir' spaces. The number of vehicle spaces needed varies due to the chance arrival of the turning vehicles and also due to the distribution of gaps of sufficient duration for the crossing to be made.

For roads with deceleration lanes in the central reserve it is not usually necessary to consider added length to the deceleration lane to cater for the wide variation in the number of vehicles waiting to turn.

For some junction designs such as 'Bulb' (Fig. 4–52(b)) or left/right staggered junctions (Fig. 4–53) the storage area is limited. These junctions should preferably be used up to the limit of capacity found by using 8 second gaps, and a storage space should be allowed for 8 vehicles. (See Fig. 4–403.) For lighter flows a minimum of 4 vehicle spaces is recommended. A length of 20 ft. for each p.c.u. may be assumed.

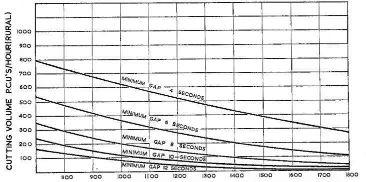


Fig. 4–403 Volume of traffic being cut p.c.u's/hour (rural)

4.407 Weaving (*Fig. 4–40(h) and (i)*). The term weaving is applied to the combined movement of merging and diverging. There are two distinct types, namely (a) weaving at roundabouts and (b) weaving along roads. In both cases there are normally other traffic streams which are non-weaving, see Fig. 4–40(h) and (i) which indicates weaving streams and non-weaving streams.

In the case of roundabouts, extensive research in this country has determined with considerable precision suitable capacity formula but in the case of weaving along roads the information available is less precise and has been derived from U.S.A. sources only: the two methods of calculation relate to different conditions. The calculation of weaving capacity along roads is of particular application to densely trafficked roads such as motorways or to roads of comparable character.

4.408 Weaving at roundabouts (*Fig. 4–40(h)*). In order to calculate the capacity of a roundabout it is necessary to prepare a diagram showing the expected future peak volumes and the proportion of weaving traffic in each weaving section. These volumes should be expressed in p.c.u's using roundabout weightings (see Table 1–42). From this diagram the proportions of weaving traffic can be read off and in conjunction with the weaving widths, entry widths and weaving lengths taken from the draft design, applied either to the formula below or to the Roundabout Design Chart Fig. 4–408 to find the practical capacity of each weaving section of the roundabout.

$$Qp = \frac{86w\left(1 + \frac{e}{w}\right)\left(1 - \frac{p}{3}\right)}{1 + \frac{w}{l}}$$

where Qp = practical capacity of weaving section of roundabout in p.c.u's per hour

w = width of weaving section in feet

e = the average width in feet of the two entry width (i.e. from an approach 'e_1' and within the roundabout 'e_2')

l = the length of weaving section between ends of guide islands.

p = proportion of weaving traffic, i.e. ratio of sum of crossing streams to the total traffic on that weaving section.

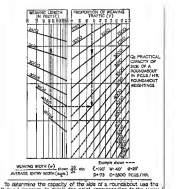
Fig. 4–40(h) shows the lengths and widths referred to.

The speed of traffic on a roundabout is affected by weaving lengths, the radius of the island, the surplus or reserve capacity, and the ratio of entry width to weaving width. As the main disadvantage of the roundabout lies in the reduction in speed of vehicles it is important to design them so that weaving lengths are longer than necessary for capacity, say 33% to 50% more. The weaving width should be 10–12 ft. wider than, but never more than double the mean entry width, and the practical capacity of the design should be well in excess of the capacity required for the flows given.

Comparison can then be made of the flow (in p.c.u's) with the capacity provided (in p.c.u's) for each weaving section of the roundabout for each peak period.

The formula given above is based on 80% of the maximum possible flows so as to provide a margin to meet the effects of wet weather, possible interaction between weaving sections, variations in flow over the hour and pedestrian influence. The formula is valid under the following conditions:

1. There are no standing vehicles on the approaches to the roundabout.
2. The site of the roundabout is level and approach gradients do not exceed 1 in 25.
3. As the formula was obtained under experimental conditions



To determine the capacity of the side of a roundabout use the left hand diagram to obtain the point corresponding to the weaving length, weaving width & average entry width (interpolating where necessary) extend a line horizontally to the right hand diagram from which the capacity is given for the proportion of weaving traffic.

To determine the dimensions of a side of a roundabout use the right hand diagram to obtain a point corresponding to capacity & proportion of weaving traffic, & draw a line horizontally to the left hand diagram. This line will cut various alternatives of weaving length, weaving width, & entry widths from which a suitable design may be obtained.

Fig. 4–408 Roundabout design diagram

(though it has since been well tried under road conditions) the ranges of variables within which the formula applies are:

$\frac{e}{w}$ $\frac{\text{average entry width}}{\text{weaving width}}$	0.4–1.0
$\frac{w}{l}$ $\frac{\text{weaving width}}{\text{weaving length}}$	0.12–0.40
p proportion of weaving traffic	0.4–1.0
l weaving length	60–300 ft.

Traffic capacity of a roundabout is affected by the geometric layout and adjustments should be made as set out below for the alignments given.

(a) Where the entry angle is between 0° and 15° deduct 5% from the capacity of the weaving section.

(b) Where the entry angle is between 15° and 30° deduct 2½% from the capacity of the weaving section.

(c) Where the exit angle is between 60° and 75° deduct 2½% from the capacity of the weaving section.

(d) Where the exit angle is greater than 75° deduct 5% from the capacity of the weaving section.

(e) Where the internal angle is greater than 95° deduct 5% from the capacity of the weaving section.

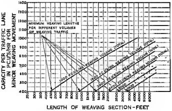
The angles referred to are those between extensions of the centre line of the weaving section and that of the approach or exit roads or the weaving section as appropriate. For geometric design see Section 4.57.

A detailed example of making roundabout calculations is given in the Memorandum on Urban Traffic Engineering Techniques Fig. 44.[a]

4.409 Weaving along roads (*Fig.* 4–40(*i*)). The effect of weaving along roads on road capacity depends upon the spacing of junctions and volumes of weaving traffic streams. To calculate the appropriate road width and spacing of junctions reference should be made to Fig. 4–409.

For given total volumes of weaving traffic the minimum length of weaving section can be read from the diagram and the lane capacity for the minor weaving movement can be determined from the left hand scale.

The number of lanes required in the weaving section can be found by summing the total of the non-weaving flow plus the major weaving flow and dividing by 1,200 and adding to this the minor weaving flow divided by the appropriate lane capacity for the minor weaving movement. The latter can be obtained from Fig. 4–409, given the weaving length and total weaving volume; see example given in the diagram. Where the calculated width is less than 3 lanes and one of the non-weaving flows exceeds 600 p.c.u's/hour an additional lane should be provided. Similarly when the calculated width is less than 4 lanes and each of the non-weaving flows exceeds 600 p.c.u's/hour, two additional lanes should be provided for non-weaving traffic.



Example:- Length between junctions—4000 feet,
Non weaving traffic—1700 PCU'S/HR,
Major weaving traffic—1450 PCU'S/HR,
Minor weaving traffic—550 PCU'S/HR,
From diagram, for total weaving volume $1450 + 550 = 2000$ PCU'S/HR, and weaving length 4000 feet, capacity per lane for minor weaving movement = 700 PCU'S/HR per lane.
∴ Number of lanes in weaving section is

$$\frac{1700 + 1450}{1200} + \frac{550}{700} = 3{\cdot}41, \text{ i.e. 4 traffic lanes.}$$

Note:- PCU'S given above are rural standard

Fig. 4–409 Capacity of long weaving sections of roads at 45–50 m.p.h. operating through speed

4.41 Signal controlled movements

For comprehensive information on traffic signal control systems reference should be made to the Manual on Roads in Urban Areas[4] and the Memorandum on Urban Traffic Engineering Techniques.[5] In some instances, particularly in semi-rural areas where signal control is applicable, junctions may be designed on channelising principles and signal control applied to simple traffic cuts. In such instances capacity checks could be made by taking the maximum flows per lane width (expressed in p.c.u's traffic signal weighting, see table 1–42) for each phase and summed for the two phases. This sum can then be compared with a practical capacity of 1,600 p.c.u's/hour per traffic lane: thus lane densities in the two phases of 800/800 1,000/600 1,200/400, etc., would be maximum practical capacities according to the proportions in the two phases.

4.42 Capacity changes from the separation of conflicts

The principle of separating traffic conflicts can be applied both to grade-separated junctions and junctions at grade. Separation of conflicts by the application of signal controls is more appropriate to urban or fringe urban areas.

The value of separating traffic conflicts can readily be shown when considering a 'T' junction but these principles apply equally to other types of junctions.

Fig. 4-42(a) shows the various merging, diverging and cutting movements which take place in a simple 'T' junction. The safety and capacity of such a junction can be improved by channelling some or all of the traffic movements to points where simpler movements can be made, thereby reducing the volume and complexity of movement at any point.

Progressive stages in effecting such changes are shown. The provision of waiting space for right-turning vehicles from the major road (b) is of first importance. The need for separate channels for either or both of the left-turning movements is not often necessary unless these volumes are high and it will seldom be found that the traffic distribution requires that separate channels are provided for both left-turning movements (c): separate channels may, however, be needed in association with acceleration and deceleration lanes.

Maximum separation of conflict and highest capacity is obtained in layouts (d) and (e), the alternative merits of which are discussed in Section 4.52 (6) and (7).

For the separation of conflicts to be effective it is important that all traffic movements shall be clearly indicated by traffic signs.

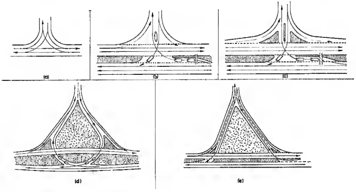


Fig. 4-42 Separation of traffic conflicts

4.50 Traffic Islands and Carriageway Markings

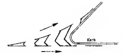
The demarcation of paths which vehicles should follow is best carried out by traffic islands and/or carriageway markings. Where islands consist only of carriageway markings they are often referred to as ghost islands. The position and shape of the islands, solid or ghost, are determined by the use of the track diagram shown in Fig. 4-34, and applications are shown in Figs. 4-50–4-53. It is important that the intersection point of the rear offside tracks of right-turning vehicles should not be more than about 5 ft. from the edge of the main carriageway into the minor road. Ghost islands or carriageway markings can extend to the edge of the vehicle paths, but a solid island should be at least 1 ft. clear of the vehicle paths, and the nose of a central dividing island should not be nearer the main carriageway than about 5 ft. The layout of a junction must be tested by using the track diagram for all turning movements. Where traffic islands are used they should be kerbed and provided with illuminated signs or bollards at suitable places, e.g. apexes to islands. Traffic signs are an integral part of the design both operationally (driver comprehension), and physically (siting, etc.).

Carriageway markings have the advantage that they can be laid down to define closely the vehicle paths and can if necessary be overrun by vehicles: they are best used:

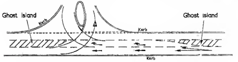
(1) to guide vehicles smoothly past traffic islands;
(2) to act as traffic islands where there is insufficient width for kerbed islands;
(3) where kerbed traffic islands are not practicable due to lack of lighting supplies;
(4) for restraining traffic to single-lane flow yet permitting room for the overtaking of a broken-down vehicle;
(5) where a kerbed traffic island would be too small to be practicable—say less than 50 square feet;
(6) to channelise vehicles in advance of a junction;
(7) as a means of trying out experimental layouts.

Examples of these uses are illustrated in Fig. 4-50. The carriageway markings should be made conspicuous by the use of reflectorised materials.

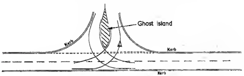
The carriageway markings may be replaced by 'joggle bars' (i.e. parallel ridges on the road surface) in cases where greater emphasis is needed.



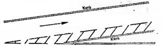
(1) Chevron marking leading to island which may be passed either side



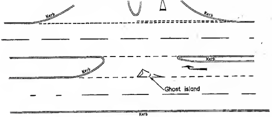
(2) Ghost islands where width is insufficient for solid islands



(3) Ghost island where there is no lighting supply



(4) Marking to restrain traffic to single lane flow with room for overtaking a broken down vehicle.



(5) Ghost island where solid island would be too small.

Fig. 4–50 Traffic islands and carriageway markings at junctions

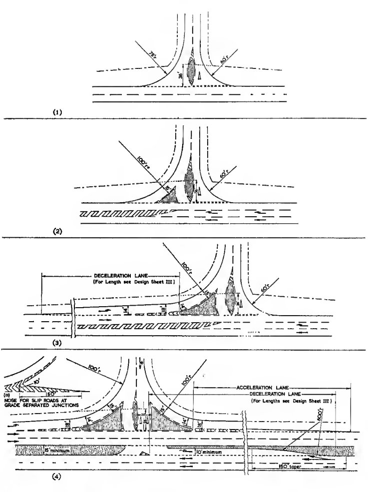


Fig. 4–51 Geometric dimensions of road junctions

4.51 Geometric Dimensions

Fig. 4–51 shows the recommended dimensions applied to junctions: these dimensions will not be given in subsequent sections and the standards to be adopted for any particular design will depend on the criteria given in Sections 4.20–4.406.

In the following sections the types of junctions are arranged from the simpler types for dealing with light traffic to the more complex. Each element of design, slip road, acceleration lane, etc., is not confined to the design in which it is illustrated and these elements should be added to or omitted from junction designs according to requirements.

4.52 T. Junctions

Figs. 4–52 refer in particular to 'T' junctions. The diagrams generally show side roads entering at right angles, but the actual angles will depend on site conditions, and some deviation from the right angle is of advantage where it assists the major flows.

(1) shows a square 'T' junction for light traffic.

(2) indicates a left-hand splay 'T' junction with a one-way slip road for traffic turning left from the major road.

(3) shows a right-hand splay junction with two-way slip roads: it will be noted that a visibility splay to the left from the minor road is required from the conflict point X (of a maximum length of 300 ft.) so that vehicles making a right turn in the approach road may have good visibility of vehicles in the main road making a fast right turn into the side road.

(4a) and (4b) shows how hatched carriageway markings may be used to define a centre lane for vehicles waiting to make right turns from the main road: this form of treatment should be considered for junctions carrying traffic volumes given by Section 4.33: for overloaded three-lane roads a layout as for a dual-carriageway road shown in (5) may be required in place of (4b).

(5) shows similar treatment applied to a dual-carriageway road: the central reserve shown has a minimum width of 15 ft. but this should be increased to 25 ft. or more whenever possible to accommodate commercial vehicles within the central width. This layout may be applied to three-lane roads by local widening. It is preferable for the carriageway area reserved for turning traffic to have a surface contrasting with the rest of the carriageway.

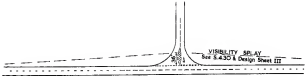
(6) shows a typical 'Bulb' junction: it has high capacity but has limitations due to reservoir space. Its layout can be varied to suit circumstances but the overall dimensions are largely controlled by the 'reservoir' space within the central reserve and this must be large enough to hold the probable number of waiting vehicles (see Section 4.406). The inset diagram (a) shows an adaptation which is suitable for 2 minor roads.

(7) shows a Delta-type junction. The alignment and dimensions can be varied to suit circumstances: suitable designs for the apex for different alignments and widths are shown in inset diagrams (a), (b) and (c). This is a high-capacity junction and the reservoir space is not so limited as in 'Bulb' junctions.

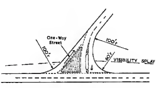
4.53 Staggered Junctions

The value of converting crossroad junctions into staggered junctions is referred to in Section 4.22: new layouts should always avoid where possible the use of uncontrolled crossroads in view of the accident risk at such junctions.

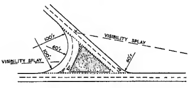
For unimportant junctions a simple right/left stagger is shown in Fig. 4–53(1). In the absence of a carriageway area for right-turning vehicles to wait, a right/left stagger is safer than a left/right and in consequence simple staggered junctions should always be of the right/left type.



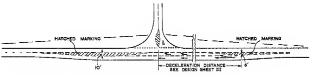
① SQUARE JUNCTION



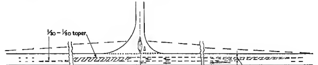
② LEFT HAND SPLAY JUNCTION



③ RIGHT HAND SPLAY JUNCTION

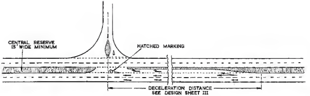


④a WIDENING TWO LANE CARRIAGEWAY FOR RIGHT TURNING TRAFFIC

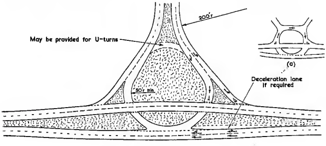


④b CARRIAGEWAY MARKING ON THREE LANE ROADS FOR RIGHT TURNING TRAFFIC

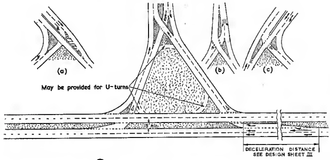
Fig. 4-52 Design of T junctions



(5) JUNCTION ON DUAL CARRIAGEWAY ROAD



(6) BULB JUNCTION



(7) DELTA JUNCTION

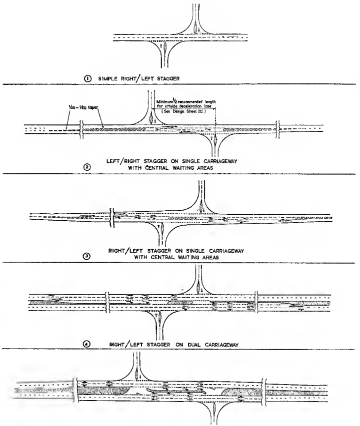


Fig. 4-53 Staggered junctions

(2) and (3) show suitable layouts for staggered junctions on single carriageways where a centre lane is reserved by carriageway markings for the use of right-turning traffic. In (3) tapers are shown to cater for the staggered movement and left-turning vehicles; in appropriate cases deceleration lanes to full standards may be provided. These layouts are applicable to 2- or 3-lane carriageways, but for two-lane carriageways local widening is necessary—see the approach road from the right in (2). The length of stagger in (2) must be sufficient to allow the reservoir space to hold the expected number of vehicles waiting to turn (See Section 4.406).

Layouts (4) and (5) are suitable for staggered junctions on dual-carriageway roads. The left/right staggers (2) and (5) can accommodate a higher volume of traffic between the minor roads but the length of stagger should be sufficient to provide for minimum offside deceleration lanes for the right turns from the major road. The layouts shown in (3) and (4) are more suitable when the right-turning traffic from the major road is of greater importance and can be achieved with the minimum stagger of 120 ft.

4.54 Y Junctions

For most types of 'Y' junctions the design can suitably follow the same features as for 'T' junctions except where the angle between two arms is very acute. In the latter case designs based on Fig. 4-54 are more appropriate for junctions of important roads. It will be noted that in this design the main flows are given the most direct paths and that where traffic cuts occur the minor traffic stream making a right turn is slowed down by curves to the stopping position. The dotted slip roads A and B for left- and right-turn movements are optional as in most of such types of layout the traffic movements for which they cater can be accommodated at other places. In some instances, where flows exceed the capacity for uncontrolled cuts the single conflict point X may be controlled by traffic signals; in this case, special attention must be paid to advance warning signs.

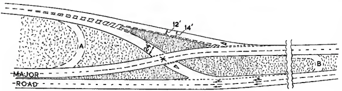


Fig. 4-54 Y Junction

4.55 Miscellaneous Complex Junctions

The general principles of separating traffic cuts can be applied to a wide variety of junctions: Fig. 4-55 shows a junction which is in principle a double 'Bulb' junction. These complex junctions require large areas of land and, in general, will only be satisfactory alternatives to staggered junctions where the existing configuration of land and roads lends itself. As this type of junction does not slow down major road traffic to the same extent as a roundabout, it may often be preferred on fast roads; roundabouts are more satisfactory where the traffic volumes on the minor roads are large.

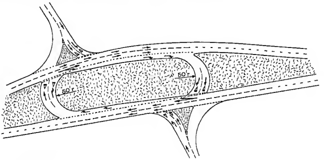


Fig. 4-55 Double junction

4.56 Crossroads

In general, uncontrolled crossroads are unsatisfactory due to the higher accident rate than that for staggered junctions: there are, however, many in existence which cannot be converted into staggered junctions: in such instances a layout such as shown in Fig. 4–56 is appropriate. A similar layout may be used for single-carriageway roads provided the road is suitably widened at the approaches to the junction: such layouts would be similar in type to Fig. 4–52 (4a) and (4b), but the central strip must be at least 15 ft. in width, preferably 25 ft., so as to accommodate most types of vehicles.

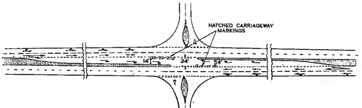


Fig. 4–56 Crossroads

4.57 Roundabouts

The geometric design of roundabouts is closely related to capacity requirements, and reference should be made to Section 4.408.

Two main considerations to be borne in mind in the design of roundabouts are:

(1) weaving sections should be adequate for capacity and designed to ensure that smooth weaving and diverging of traffic takes place.

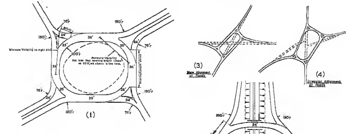


Fig. 4–57 Design of roundabouts

(2) the traffic streams within the roundabout are dominant over the entry streams.

These main requirements can be met in the following ways:

(a) Direct traffic cuts should be prevented by making the ratio of weaving length to weaving width large enough: a ratio of 4:1 should be regarded as a minimum.

(b) Entry radii should be less than the radii within the roundabout: for the entry radii the minimum should be 60 ft. and desirably 80 to 120 ft., for radii on the roundabout the minimum should be 80 ft., and in other cases about a third greater than entry radii.

(c) Entry angles should be larger than exit angles, and it is desirable that the entry angles should be about 60° if possible: the exit angles should be small, even tangential.

(d) The lengths of arc on the approaches (at the entry radii given by (b) above) should be long enough to ensure that the radii of the paths taken by entering vehicles are less than those of vehicles on the roundabout.

(e) The external kerb line of weaving sections should not normally be re-entrant but consist of a straight or large radius curve of the same sense as the entry and exit curves.

(f) The weaving width should normally be one traffic lane wider than the mean entry width thereto.

(g) Widening on curves is recommended to assist the average vehicle; an appropriate amount of widening for each traffic lane is 2 ft. for 50 ft. radius, 1 ft. for 75 ft. radius, and 6 in. for 100 ft. radius.

(h) It is of advantage, particularly on narrow entry approaches, to bellmouth the entry width.

The effect of the priority rule at roundabouts on their design is still being studied. There are indications that capacities may be satisfactorily increased by providing larger weaving widths than indicated in (f) above by reducing the size of the central islands. Accordingly, in suitable instances, some consideration should be given to the use of smaller central islands, particularly where they may be tried before making permanent.

Fig. 4–57(1) shows an idealised design where entry angles are 60° and exit angles 30°. This design can only be achieved by staggering the approach roads.

For most situations roundabouts will need to be 'tailor made' to suit traffic flows and site conditions, and the suitable design can only be found from detailed calculations for capacity by trial and error.

Fig. 4–57(2) illustrates the different peak hour flows which occur and the need that is usually found for longer weaving lengths and higher capacities for opposite sides of a roundabout, with the result that rectangular or oval shapes are usually more appropriate than squares or circles.

Figs. 4–57(3) and (4) show how approach roads need to be re-aligned where they cut at skew angles or meet at irregular angles.

The capacity and operational characteristics of roundabouts can be greatly improved by the use of grade separation for one or both of the through traffic flows. This results from the reduction in the volume of weaving traffic as well as from the segregation of the faster through traffic from the turning traffic.

As a principal difficulty associated with roundabouts is that of inducing drivers to slow down to suitable speeds, it is therefore important that adequate traffic signs are provided for this purpose: "Reduce Speed Now" signs of suitable size, properly sited in advance of roundabouts should be used for this purpose.

Fig. 4–57(5) shows the detailed aligement of a roundabout with grade separation for one road: fully grade-separated designs will be discussed in Sections 4.60–4.70. For such roundabouts the large size leads to high operating speeds, and it is important that the various elements, entry radii, roundabout radii, and lengths of weaving sections are designed to suit these speeds.

Grade Separated Junctions

4.60 General

The conditions where the use of grade separation is warranted are usually as follows:

(a) A ground level scheme of sufficient capacity is not practical: see Sections 4.12 for general information and 4.40 for capacity calculations.
(b) The scheme is justified economically from the saving in delay to traffic and accidents: see Section 4.12 and also the memorandum on Urban Traffic Engineering Techniques.[6]
(c) Grade separation is cheaper on account of topography or on grounds that expensive sites can be avoided by it.
(d) For operational reasons; see Section 4.11.
(e) Where roads cross motorways.

The use of grade separation may range from separating one traffic movement from all other traffic movements to the complete separation of each traffic movement from every other movement so that only merging or diverging movements remain. Where some or all turning movements can be catered for elsewhere, a simple flyover with few or no connections may suffice. The extent to which individual traffic movements should be separated from other traffic movements depends on capacity requirements and these should be calculated: it also depends on the extent to which important traffic movements should be given free flow conditions, e.g. where a particular movement has a volume approaching the capacity of a single traffic lane.

4.61 Traffic Diagrams

In order to select the most appropriate designs a study will be needed to determine the estimated future heaviest traffic flow conditions (see Sections 1.40 and 4.10). Information will be required of all directional traffic movements which will occur under these conditions. For all but the simplest types of junctions it is usually advantageous to set down the future peak traffic distribution in a table: Fig. 4.61 shows typical examples. It may be necessary in some instances to check whether at times other than peak hours combinations of flow occur which exceed those obtaining at peak times.

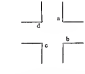
4.62 Site Conditions

A layout plan should be available showing ground levels, any 'fixed' road alignments, the position of buildings, etc., so that the extent of land available and topographical features can readily be seen. It is helpful if 'soft' (easily and cheaply available) and 'hard' (difficult to acquire or expensive) properties are indicated so that the type of layout chosen can be such as to minimise the cost and difficulty of acquiring property. It is advantageous for land values to be indicated on these plans so that land costs for a given scheme can be determined.

4.63 Choice of Scheme

From a study of the traffic conflicts it will generally be apparent which traffic streams must be grade-separated, leaving the other streams to be dealt with by junctions at grade; the choice of these will depend upon the capacities needed. A study of the characteristics of various types of grade-separated junctions is necessary, and a number of alternative designs should be prepared. The final choice of scheme must satisfy capacity requirements, geometric standards, operational needs and economical design. It will be necessary to have regard to the practicability of maintaining traffic during construction and in some instances the choice of a particular design will be determined by the adoption of two-stage construction, e.g. constructing a roundabout first and providing grade separation later.

1. Four Way Junction



Road Approach	Traffic volumes in p.c.u's/hr. (rural standard)		
	L (left)	S (straight)	R (right)
a			
b			
c			
d			

2. Multiway Junction

Traffic volumes in p.c.u's/hr. (rural standard)						
To From	a	b	c	d	e	f
a						
b						
c						
d						
e						
f						

Note: It is usually convenient to give the volumes at each peak period of the day, indicating say the evening peak flow by bracketed figures.

Fig. 4-61 Traffic diagrams for junctions

4.64 Special Geometric Standards

Whilst the geometric standards given for roads and junctions also apply to grade-separated interchanges, the low design speeds, usually 30 m.p.h., of slip roads, loops, roundabouts, and other ancillary roads, necessitate further standards to be given.

4.641 Visibility. Stopping sight distances of 200 ft. for the recommended 30 m.p.h. design speed should always be provided between points 3 ft. 6 in. above the centre of the lane on the inside of bends; where the design is sub-standard the appropriate stopping sight distances are 150 ft. for 25 m.p.h. and 110 ft. for 20 m.p.h.

The visibility standards for junctions and roundabouts are given in Sec. 4.30.

4.642 Acceleration and Deceleration Lanes. The standards for speed-change lanes are given in Secs. 4.31–4.33. As traffic speeds and volumes are normally high on roads with grade separation it is important that the full standards are met.

4.643 Horizontal Curves and Superelevation. Appropriate radii for different design speeds are given in Sec. 4.34 and Design Sheet IV. As the normal design speeds for loops and slip roads will be 30 m.p.h. the radius should not be less than 200 ft. with maximum superelevation of 7% (1 in 14½). If smaller radii are unavoidable, warning signs will be necessary. For curves up to 650 ft. radius the maximum superelevation should be provided: for greater radii the superelevation should be appropriate for a 40 m.p.h. Design Speed—See Diagram 3-24, Superelevation of curves.

It is important where transitions occur from high to low speed that the curves should be compound or transitional, the radius at any point being appropriate for the vehicle speed at that point.

4.644 Vertical Curves. To ensure reasonable standards of visibility, comfort and appearance, vertical curves should be introduced at all changes in gradients. For 30 m.p.h. design speed a curve to give the stopping sight distance of 200 ft. should be regarded as a minimum for one-way roads or two-way roads with added width for passing: for comfort a recommended K value (see Sec. 3.27) is 20. For design speeds of 25 and 20 m.p.h. stopping sight distances are 150 and 110 ft. respectively and the corresponding recommended K values 15 and 10. The minimum length of vertical curve should not be less than 3V feet where V is miles per hour.

4.645 Width of Slip Roads, etc. Widths of slip roads should be in accordance with Sec. 4.35 and Design Table IV. Where slip roads need to be restricted to one lane, for example where joining an acceleration lane, the width sufficient for passing a broken down vehicle should be retained and the width restriction effected by the use of carriageway markings—see Sec. 4.50.

4.646 Gradients. The normal maximum gradient of 1 in 25 (4% grade) given in Sec. 3.28 may be exceeded for slip roads but an up gradient of 1 in 20 (5%) and a down gradient of 1 in 15 (7% approximately) should normally be regarded as maxima.

4.647 Vertical Clearances. The vertical clearances—see Sec. 3.30—should be maintained on slip roads.

4.65 Design Principles

Special design principles apply to interchanges and must be considered when comparing the characteristics of alternative designs.

4.651 The high speeds normally met with on roads where grade separation is applied and the low design speeds of ancillary roads, etc., make it necessary to pay particular attention to the problem of transitions between high and low speed. This not only influences the use of long speed-change lanes and compound curves but also the choice of types of interchange which do not necessitate abrupt changes in the speed of vehicles.

4.652 Weaving on the main carriageways within the interchange is undesirable and can be avoided by arranging for diverging points to precede merging points, this can often be achieved by the use of ancillary roads.* Small weaving flows (up to 1,000 p.c.u's/hour) taking place on the main carriageway are permissible only where an extra traffic lane is provided.

4.653 Access connections to major roads should be kept to a minimum.

4.654 On a road with a large number of grade-separated interchanges, a consistent design speed is desirable for slip roads, loops and ancillary roads.

4.655 As a general rule right-turning movements which are grade-separated should be made through a left-hand slip road; divergent forks on the major road should be avoided where the left-turning movement is of greater volume than the right-turning movement.

4.656 Unexpected prohibited traffic movements, especially where traffic is light, are difficult to enforce and cause danger. If possible the geometric layout should be designed to make the prohibited movements difficult, e.g. on one-way slip roads entry contrary to the one way movement can be restricted by the use of suitably shaped islands to supplement the traffic signs.

4.657 The path which a driver should take should be readily comprehensible and unnatural operational movements should be avoided as far as possible. For instance, drivers expecting a simple left turn may be confused by having to make a right turn first. Other considerations such as minimisation of traffic conflicts and easy exit radii may take precedence.

4.658 The distance between merging and/or diverging points on ancillary roads should be sufficient for drivers to make appropriate decisions; this distance should, if practicable, be not less than that travelled in 6 seconds at the design speed, e.g. about 250 ft. at 30 m.p.h.

4.659 On roads where grade separation is required at most junctions, it will usually be necessary to restrict right-turning movements at access points along the roads, and provision is then needed for 'U' turns at interchanges.

4.66 Types of Junctions

The line diagrams illustrate some of the basic types of junction but there are many possible variations. A number of types of junction have purposely been omitted where certain features of the layouts are not in the most satisfactory form, for example, some layouts where right turns are made from the fast (offside) traffic lane or where the 'hand' of the layout restricts the radius of entry. This does not, however, preclude the adoption of such layouts for special site conditions.

4.67 Three-way Junctions

Fig. 4.67(a) shows a typical 'trumpet' or 'jug-handled' junction; it can be designed of opposite 'hand' but that shown is to be preferred as it does not require such a severe reduction in speed when leaving the major road, and the slightly skew alignment shown has some advantages over right-angled alignment. For some 'Y' junctions where traffic flows between the acute-angled approaches are small and can be diverted elsewhere, grade

*Sometimes referred to as CD roads (see example in Fig. 4.69(e)).

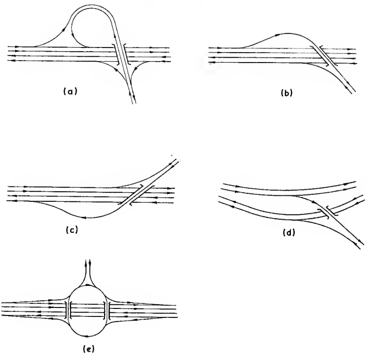


Fig. 4-67 Three-way junctions

separation of only one traffic stream is needed, and Figs. 4–67(b), (c) and (d) show typical layouts. The layout in Fig. 4–67(d), although showing the less satisfactory right turn movement from the offside lane, may still be acceptable where the divergent flows are almost equal, or as an isolated grade-separated junction on a road where most junctions are at grade. Where three important roads meet it may be necessary to adopt a layout with a 3-level bridge or 3 separate bridges.

Where a three-way junction is likely to become a four-way junction eventually, a design should be chosen which is readily adaptable, e.g. 4–67(c) to 4–69(a), or 4–67(a) to cloverleaf or part cloverleaf design.

4.68 Junctions of Major/Minor Roads

For a junction between an important road and a less important one it will usually be satisfactory to permit traffic conflicts to take place on the minor road. Except for very light flows, the normal diamond layout should be modified as in Fig. 4–68(a) which allows suitable spaces for vehicles waiting to turn right into the slip roads without adding to the width of the bridge structure. It is important that on one-way slip roads entry contrary to the one-way movement should be prevented, not only by signs but also by the layout (See inset diagram).

An alternative design is the half-cloverleaf, types of which are shown in Figs. 4–68(b), (c), (d) and (e). The half-cloverleaf has the advantage that it can often meet difficult site conditions. The slip roads can be placed on the opposite 'hand' if it is necessary to minimise right turn cutting movements on the minor road.

The form that a half cloverleaf takes will depend primarily upon the importance of various traffic movements, and loops and slip roads should be sited to minimise right turn conflicts. A number of variations are possible for meeting different site conditions and traffic distribution. Where the volume of traffic justifies it, however, one or more extra slips roads can be provided as in Figs. 4–68(d) and (e).

It is to be stressed that in the hand shown in Fig. 4–68(b), the slip road from the major road does not require such a sudden reduction in speed as the loop in (c) and it is therefore to be preferred in the absence of other overriding factors.

The uncontrolled junctions within the interchange should be designed for capacity and safety in accordance with Sections 4.10–4.57.

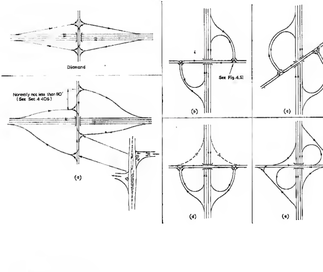


Fig. 4.68 Junctions of major/minor roads

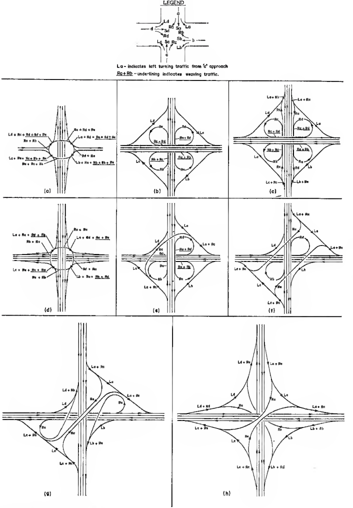


Fig. 4-69 Junctions of two major roads

4.69 Junctions of two Major Roads

This class of interchange is concerned with layouts where all traffic cuts are eliminated, but can only be justified where high traffic flows would make the use of simpler types unsatisfactory. The choice of the type of interchange for a particular site depends upon the configuration of the site and the traffic volumes and distribution: in many instances expensive land can be avoided by the choice of a suitable design.

The most suitable type of interchange for a particular site can be determined by applying the traffic data to a number of likely schemes and selecting from these the schemes which adequately meet traffic requirements. The schemes then need to be tested against site conditions and alternative costs compared.

Fig. 4–69 illustrates a number of basic designs arranged roughly in order from the simple and less costly to the more elaborate and expensive; in the simpler schemes some traffic conflicts are dealt with by the principle of weaving, and in others traffic conflicts are eliminated by grade separation. Though not to scale, the diagrams indicate broadly the relative sizes of different types of interchange.

In the diagrams each traffic movement is indicated by appropriate letters on each slip road, loop or other ancillary road so that by reference to a traffic diagram actual traffic volumes may be substituted.

Figs. 4–69(a)–(d) show standard types of roundabout and clover leaf interchanges. The advantages and disadvantages are set out below.

Advantages	Disadvantages
(a) *Roundabout with grade separation for one major road.*	
(i) Occupies comparatively small overall area. (ii) Requires less carriageway area than other grade separated interchanges. (iii) Allows for easy 'U' turns. (iv) Suitable for most sites.	(i) Straight through traffic on one road required to weave with turning traffic from other road. (ii) Capacity is limited.
(b) *Simple cloverleaf.*	
(i) Through traffic on both major roads is unimpeded. (ii) Only one bridge is required. (iii) Left-turning traffic has a direct path.	(i) Occupies a large overall area. (ii) Requires greater carriageway area than (a) and (d). (iii) Bridge is more costly than (a). (iv) Weaving capacity is limited to 1,200 p.c.u./hour (but may be increased, see (c)). (v) 'U' turns are long and operationally difficult. (vi) Four connections on each main carriageway. (vii) In one weaving section both weaving flows will be peak flows at the same time. (viii) Right-turning traffic has to reduce speed severely to negotiate loops.
(c) *Cloverleaf with low speed ancillary roads for weaving traffic.*	
Same as (b), (i), (ii), (iii) and: (iv) Weaving capacity increased by ancillary roads (for two-lane roads about 2,000 p.c.u./hour). (v) Number of connections to major roads reduced as compared with (b).	Same as (b), (i), (ii), (iii), (v), (vii), (viii), but: (i) Even more space and carriageway area required, and (ii) Bridging is still more expensive.
(d) *Roundabout with grade separation for both major roads.*	
(i) Through traffic on each major road is unimpeded. (ii) Occupies a small overall area and requires less carriageway area than a cloverleaf. (iii) Weaving volumes are smaller than in (a). (iv) Compared with the cloverleaf, at peak times one weaving flow only in each weaving section will be a peak flow.	(i) The layout is three-level. (ii) Bridging costs are high and similar to (c).

It will be seen from the above notes that the choice will normally lie between types (a) and (d); types (b) and (c) may be found applicable for special site conditions, e.g. where land is cheap and where traffic flows favour such layouts.

Where weaving movements are too great to be catered for by any of the foregoing, an alternative is to grade separate one or more right-turning movements as is shown in Figs. 4–69(e), (f) and (g). Other solutions involving the same principles are possible and may be necessary in particular site or traffic conditions. Fig. 4–69(f) shows a layout suitable for a site where land is restricted on opposing quadrants. Fig. 4–69(h) shows a case where all turning movements are catered for by direct non-weaving paths with only two connections to each carriageway. Expense makes it suitable only in exceptional cases.

It should be noted that in the types of interchange shown in Figs. 4–69(e), (f), (g) and (h), 'U' turns are not possible (except for two directions in type (e)).

4.70 Complex Grade-Separated Interchanges

Interchanges of more than two major roads designed to eliminate most weaving movements are necessarily complicated in design, operationally difficult, occupy large areas of land and, requiring numerous bridges, are costly to construct.

In most instances the traffic needs may be met by combinations of roundabouts and grade separation, the latter being applied to the more important traffic movements. It is preferable, where possible to change the road layout to simplify the traffic pattern; for example a 5-way interchange might be resolved into a 3-way and a 4-way interchange spaced adequately to cater for weaving traffic between them.

5 Rural Motorways

5.10 General

This section deals with capacity and road geometry only; other aspects given in sections 1–4 are not necessarily applicable to motorways.

The standards given in sections 1–4 relating to all-purpose dual carriageway roads with a Design Speed of 70 m.p.h. also apply to motorways generally. The higher mean speed of vehicles on motorways, resulting from the complete control of access, and the prohibition of usage by pedestrians and specific classes of vehicles, do not generally permit the relaxation of requirements permissible under certain circumstances for all-purpose roads. In some respects higher standards are required as discussed in this section. The cost implications of adopting extra high standards must, however, be carefully considered.

This section follows the same order as sections 1–4 (relevant section numbers are given in brackets) setting out the main differences and standards to be followed in the design of motorways.

5.11 Road Siting and Amenities (2.2)

Objectives in road alignment generally are listed below:

(i) Care should be taken to ensure that embankments and cuttings do not make severe breaks in the natural skyline.

(ii) When negotiating a ridge in cutting or travelling through a broad stretch of woodland, the road should be on a curve wherever possible, so as to preserve an unbroken background.

(iii) Short curves and straights should not be used; horizontal and vertical curves should be as long as possible; adjacent curves should be similar in length.

(iv) Small changes of direction should not be made, as they give the perspective of the road a disjointed appearance.

(v) To relieve the monotony of driving on a long straight road it is an advantage if it can be sited to give a view of some prominent feature ahead.

(vi) Curves of the same or opposite sense which are visible from one another should not be connected by a short straight. It is better to introduce a flat curve between curves of the same sense, or to extend the transition curves to a common point between curves of the opposite sense.

(vii) Changes in horizontal and vertical alignment should be phased to coincide wherever possible. Phasing becomes increasingly important when using horizontal curves sharper than 7,000 ft. radius and vertical curves of less than 50,000 ft. radius.

(viii) Flowing alignment can most readily be achieved by using long curves, in preference to straights.

(ix) The profile of the road over bridges must form part of the easy flowing alignment.

(x) As stated in Section 5.15 (3.24) great care should be taken to obtain smooth, flowing edge profiles when applying or removing superelevation.

(xi) Sharp horizontal curvature should not be introduced at or near the top of a pronounced summit curve. This condition is hazardous, especially at night, in that the driver cannot see the change in horizontal alignment.

(xii) To avoid giving a distorted appearance to the view of the road ahead, sharp horizontal curvature should not be introduced at or near the low point of a dip.

(xiii) Where horizontal curves start near summits or dips, care should be taken when introducing superelevation to avoid creating large flat areas on which water would stand.

(xiv) Horizontal and vertical curves should be made as flat as possible at intersections and in order to give generous sight distances.

5.12 Road Cross Section (3.10–3.15)

The cross section of motorways can be standardised due to the absence of variables to cater for the many conditions applying to all purpose roads, and Fig. 5–12 gives the normal cross section for a motorway. Central reserves should be provided to the standard width of 13 ft. Exceptionally the width may be reduced to 8 ft. on long bridges or viaducts.

5.13 Carriageway Capacities (3.20)

The standard design capacity for all purpose roads also applies to long motorways, but as higher capacities with consequently reduced speeds are more acceptable over shorter journeys it is permissible to increase design capacities where the average journey lengths along a road are shorter. Table 5–13 below gives recommended standards.

Standard 1 applies to the best operating conditions, Standard 2 to Sections of road where average journey lengths are less than 25 miles, and Standard 3 is acceptable where average journey lengths are below 10 miles.

The average journey length may be estimated either from Origin and Destination Surveys or from the proportion of traffic entering and leaving the motorway at junctions.

These standards may also be applied to all purpose roads of high quality with grade-separated junctions.

The use of directional peak hour capacities in the table below is to be preferred to daily capacities where adequate traffic data is available.

Table 5–13

Standard	Average Journey length in miles	Daily Capacities (16 hr. August day in p.c.u's.)			Peak Hourly Capacity per lane/ p.c.u's.
		Dual 2 lane	Dual 3 lane	Dual 4 lane	
1	Over 25	33,000	50,000	66,000	1,000
2	10–25	40,000	60,000	80,000	1,200
3	Under 10	50,000	75,000	100,000	1,500

These standards should in the main be used for checking whether on particular sections of a route the width chosen for the route generally (usually Standard 1) is adequate for the more densely trafficked sections along it and these must comply with Standards 2 or 3 as appropriate.

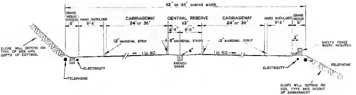


Fig. 5–12 Typical cross section of rural motorway

5.14 Horizontal Curvature (3.22–3.24)

In order to obtain the stopping sight-distance of 950 ft. within the highway boundaries the radius of curvature will need to be greater than 5,000 ft.; at bridges, etc., where a fixed object is 3 ft. from the marginal strip the radius requires to be over 11,300 ft. The difficulties arising from low-radius curves and obstructions such as bridges need special consideration at the initial location stage. It is also to be noted that on rising gradients left-hand low-radius curves give rise to more accidents.

It may be necessary at sites where low-radius curves occur at bridges to compare the costs of alternative schemes providing for the required visibility. At sites of particular difficulty when no economic alternative exists, then some relaxation on the nearside may be considered within the following limits:

(a) There must be 950 ft. visibility behind any fixed obstruction on the nearside.
(b) Visibility in front of the obstruction must not be less than 700 ft.
(c) The width of obstruction must not exceed 3 ft.
(d) The length of obstruction must not exceed 60 ft.
(e) There must not be more than one such obstruction within the sector enclosed by a 950 ft. chord.

5.15 Superelevation (3.24)

Where the development of superelevation or the removal of adverse camber is carried out by pivoting the road about the line of the nearside lane edge of the carriageway it is desirable that the difference in grade between the nearside and offside edge of each carriageway should not exceed 0.5 % and adequate vertical curves should be provided.

5.16 Transition Curvaa (3.25)

For aesthetic reasons consideration should be given to the use of transition curves on radii up to 10,000 ft. where practicable, but transitions must be provided on curves the radius of which is less than those given in Column 8, Design Table II.

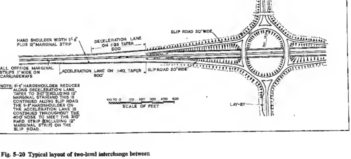


Fig. 5–20 Typical layout of two-level interchange between rural motorway and all-purpose road

5.17 Vertical Curves (3.27)

Where economically practicable the K values for crests should be increased, preferably to 500.

5.18 Gradients (3.28)

For motorways a gradient of 3% should be regarded as the normal maximum but in hilly country 4% is acceptable as a limiting gradient. When considering additional ascending lanes on long hills the permissible overloading should be limited to 50% for Standard 1, 30% for Standard 2 and 15% for Standard 3 of the capacities given in the table to 5.13.

5.19 Grade Separated Junctions (4.60)

The spacing of junctions should be as long as compatible with the aim to attract through traffic to the motorway system. In general they should not be spaced closer than 10 miles apart in sparsely populated areas but in more developed areas closer spacing will be necessary. The minimum spacing in rural areas should not normally be less than 3 miles: at 2 miles spacing advance signing becomes difficult.

5.20 Slip Roads (4.645)

For motorway junctions, slip road widths of 20 ft. will normally be appropriate, with added width for curvature. At acceleration and deceleration lanes the 20 ft. width should be provided at the end of the taper in accordance with Fig. 5–20. The hard shoulders on the motorway should not be continued along the slip roads, but should be terminated as shown in Fig. 5–20. The provision of lay-bys at motorway junctions is a desirable feature, and they should if possible be sited on the all-purpose road, on each side of, and as near as practicable to the junction. Fig. 5–20 shows the recommended siting.

5.21 Geometric Standards for Roads in Junctions (4.64–4.644)

Design speeds should if practicable exceed those for all purpose roads (30 m.p.h.) and 40 or 50 m.p.h. should be adopted if appropriate for the layout (sections 3.22, 3.24, 3.27 and Design Table II give standards for 40 and 50 m.p.h.). In particular, stopping sight distances for horizontal and vertical curvature should if possible comply with 50 m.p.h. design speeds.

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MINISTRY OF TRANSPORT
SCOTTISH DEVELOPMENT DEPARTMENT
THE WELSH OFFICE

Advisory Manual The Layout of Roads in Rural Areas

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Foreword
by the Minister of Transport
The Rt. Hon. R. Marsh, M.P.

Everybody knows about the growth of traffic and how this highlights the need for safe, well designed roads or junctions built with an eye for future needs. Some people can do something about making sure that roads are so designed—and as Minister of Transport, it is part of my job to help them.

This manual is part of that help. It is intended for highway engineers who need a high degree of skill in forecasting future traffic requirements and, remembering that resources are limited, in selecting the means that meet those requirements in the most economic way.

The manual uses the most up-to-date information from research and experience: it gives guidance on the standards required for road safety and road and junction capacity. I hope it will help in removing uncertainties in design and in ending the waste of resources which result from either over or under design.

During the last three years my Ministry has published Urban Traffic Engineering Techniques and Roads in Urban Areas for highway engineers in urban areas. This new manual (and its companion volume Traffic Prediction for Rural Roads) will, I hope, prove equally valuable for those concerned with rural roads.

Contents

Figures

Tables

Introduction

This memorandum deals with the principles for the design of rural roads and junctions. Whilst it supersedes Memoranda No. 575 Layout and Construction of Roads and No. 780 Design of Roads in Rural Areas most of the geometric standards contained therein have not warranted any change. It is, however, more comprehensive and completely revises the standards for carriageway capacity.

Standards for junction design and capacity calculations for junctions form a new feature included in this memorandum. The memorandum has been prepared for the Highway Standards Committee of the Ministry of Transport by a small working party representing the County Surveyors' Society, the Road Research Laboratory and the Ministry of Transport.

The Working Party wish to record the helpful advice received from the parent bodies with whom they have liaised throughout the preparation of the memorandum.

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